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S.M. Jones

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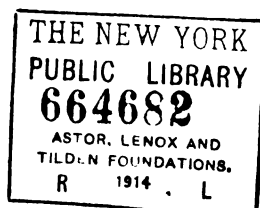
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GRAPHIC STATICS
STRESSES IN BRIDGE TRUSSES
BRIDGE MEMBERS AND DETAILS
BRIDGE TABLES

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PRINTED IN THE UNITED STATES

BURR PRINTING HOUSE
FRANKFORT AND JACOB STREETS
NEW YORK



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PREFACE

The International Library of Technology is the outgrowth of a large and increasing demand that has arisen for the Reference Libraries of the International Correspondence Schools on the part of those who are not students of the Schools. As the volumes composing this Library are all printed from the same plates used in printing the Reference Libraries above mentioned, a few words are necessary regarding the scope and purpose of the instruction imparted to the students of ~~and the class of students taught by~~—these Schools, in order to afford a clear understanding of their salient and unique features.

The only requirement ~~for admission~~ to any of the courses offered by the International Correspondence Schools, is that the applicant shall be able to read the English language and to write it sufficiently well to make his written answers to the questions asked him intelligible. Each course is complete in itself, and no textbooks are required other than those prepared by the Schools for the particular course selected. The students themselves are from every class, trade, and profession and from every country; they are, almost without exception, busily engaged in some vocation, and can spare but little time for study, and that usually outside of their regular working hours. The information desired is such as can be immediately applied in practice, so that the student may be enabled to exchange his present vocation for a more congenial one, or to rise to a higher level in the one he now pursues. Furthermore, he wishes to obtain a good working knowledge of the subjects treated in the shortest time and in the most direct manner possible.

In meeting these requirements, we have produced a set of books that in many respects, and particularly in the general plan followed, are absolutely unique. In the majority of subjects treated the knowledge of mathematics required is limited to the simplest principles of arithmetic and mensuration, and in no case is any greater knowledge of mathematics needed than the simplest elementary principles of algebra, geometry, and trigonometry, with a thorough, practical acquaintance with the use of the logarithmic table. To effect this result, derivations of rules and formulas are omitted, but thorough and complete instructions are given regarding how, when, and under what circumstances any particular rule, formula, or process should be applied; and whenever possible one or more examples, such as would be likely to arise in actual practice—together with their solutions—are given to illustrate and explain its application.

In preparing these textbooks, it has been our constant endeavor to view the matter from the student's standpoint, and to try and anticipate everything that would cause him trouble. The utmost pains have been taken to avoid and correct any and all ambiguous expressions—both those due to faulty rhetoric and those due to insufficiency of statement or explanation. As the best way to make a statement, explanation, or description clear is to give a picture or a diagram in connection with it, illustrations have been used almost without limit. The illustrations have in all cases been adapted to the requirements of the text, and projections and sections or outline, partially shaded, or full-shaded perspectives have been used, according to which will best produce the desired results. Half-tones have been used rather sparingly, except in those cases where the general effect is desired rather than the actual details.

It is obvious that books prepared along the lines mentioned must not only be clear and concise beyond anything heretofore attempted, but they must also possess unequalled value for reference purposes. They not only give the maximum of information in a minimum space, but this information is so ingeniously arranged and correlated, and the

PREFACE

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indexes are so full and complete, that it can at once be made available to the reader. The numerous examples and explanatory remarks, together with the absence of long demonstrations and abstruse mathematical calculations, are of great assistance in helping one select the proper formula, method, or process and in teaching him how and when it should be used.

This volume contains a thorough treatment of graphic statics, a subject that is indispensable for the determination of stresses in structures; a complete but simple exposition of the methods used in computing the stresses in the members of all common types of bridge trusses; a full description of all the structural shapes used in bridge constructions; and extensive tables where the properties—areas, radii of gyration, weights, dimensions, etc.—of these shapes are given. After the general methods of determining stresses have been presented, trusses of various types (Pratt truss, Howe truss, Warren truss, Baltimore truss, Petit truss, curved-chord trusses, etc.) are taken in succession; and the stresses in them computed, by either the graphic or the analytic method, that method being selected which is best adapted to each particular case.

The method of numbering the pages, cuts, articles, etc. is such that each subject or part, when the subject is divided into two or more parts, is complete in itself; hence, in order to make the index intelligible, it was necessary to give each subject or part a number. This number is placed at the top of each page, on the headline, opposite the page number; and to distinguish it from the page number it is preceded by the printer's section mark (§). Consequently, a reference such as § 16, page 26, will be readily found by looking along the inside edges of the headlines until § 16 is found, and then through § 16 until page 26 is found.

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GRAPHIC STATICS

COPLANAR CONCURRENT FORCES

1. The Graphic and the Analytic Method Compared.—The graphic method of dealing with problems in statics, although not so accurate as the analytic, is in a great many cases much simpler than the latter and gives results that are sufficiently close for all engineering purposes. It is not, however, to be supposed that the graphic method is preferable in all cases; for the constructions it requires are sometimes of exceeding complexity compared with the simple formulas employed by the analytic method. Moreover, the latter method is broader and better adapted to discussions and investigations of a general character.

As an illustration, let it be required to find the equilibrant of two forces F_1 and F_2 , Fig. 1, acting at a point A in given directions. From A (or from any other point) two lines are drawn parallel, respectively, to the given directions of the forces. On these lines, and in the proper directions, are laid off the vectors AB_1 and AB_2 , to represent F_1 and F_2 , using any convenient scale. On the lines AB_1 and AB_2 the parallelogram AB_1BB_2 is constructed, whose diagonal BA is the required equilibrant. Its magnitude is determined by measuring the line BA and multiplying its length by the scale used. Thus, if F_1 and F_2 are laid off to a scale of 100 pounds to the inch, and BA measures $3\frac{7}{16}$ inches, the magnitude of the equilibrant Q is

$$3\frac{7}{16} \times 100 = 344 \text{ pounds, nearly.}$$

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Instead of constructing the whole parallelogram, the triangle AB , B may be constructed, by drawing B_1B equal and parallel to F_1 , and then drawing BA .

Here the graphic solution is evidently simpler and can be accomplished much more rapidly than the analytic solution.

2. Suppose, now, that a weight of 1,000 pounds, Fig. 2, is placed on a beam A_1A_2 , at distances of 9 and 3 feet from the supports A_1 and A_2 , and that it is required to find the reactions at those points. Graphically, the problem may be solved as follows: From A_1 , draw A_1W' to represent the weight of 1,000 pounds to any convenient scale; draw $W'A_2$, the vertical A_1B_1 , and the horizontal B_1B_2 . Then will AB ,

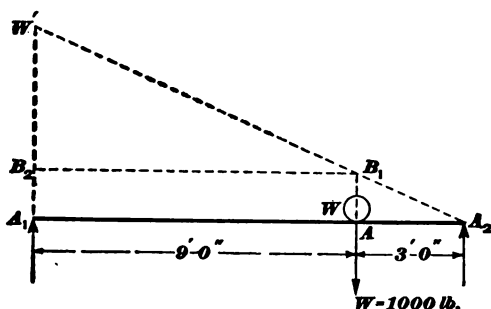


FIG. 2

(or A_1B_1) represent the reaction R_1 at A_1 , and B_1W' the reaction R_2 at A_2 . For, if moments are taken about A_2 , we must have,

$$R_1 \times A_1A_2 = W \times AA_2, \text{ or } \frac{A_1A_2}{AA_2} = \frac{W}{R_1}$$

Now, the similar triangles A_1A_2W' and AA_2B_1 give

$$\frac{A_1A_2}{AA_2} = \frac{A_1W'}{AB_1}$$

Comparing this with the preceding proportion and noticing that $A_1W' = W$, it follows that $AB_1 = R_1$. We must have, also, $R_2 = W - R_1$. The figure gives,

$$B_1W' = A_1W' - A_1B_1 = A_1W' - AB_1 = W - R_1 = R_2$$

In this case, the geometric construction, although simple, can be advantageously dispensed with, as the reactions can be much more readily calculated from the general equation

of equilibrium referred to above; viz., $12 R_1 = 3 W$; whence, $R_1 = \frac{1}{4} W = 250$ pounds, and $R_2 = W - R_1 = 1,000 - 250 = 750$ pounds.

This example shows that the graphic method is not always the simpler, and that some judgment should be exercised in the choice of the method to be employed in any particular case. There are geometric constructions by which centers of gravity, statical moments, bending moments, and shearing stresses (terms used in the mechanics of materials) can be determined. But, unless these constructions lead to the required results more easily and quickly than the formulas employed by the analytic method, the formulas are to be preferred.

3. Graphic Determination of the Resultant and Equilibrant of Any Number of Concurrent Forces.

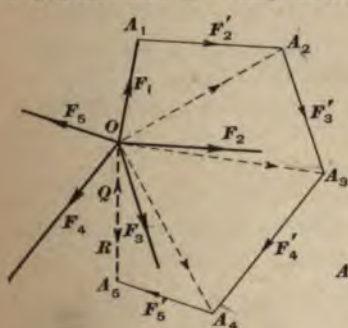


FIG. 3

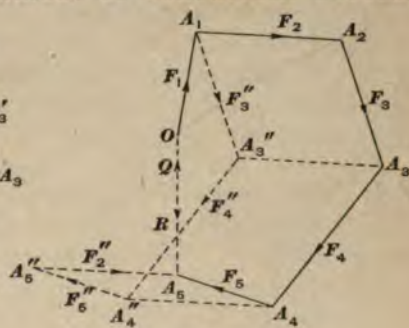


FIG. 4

The construction to be presently described has already been explained, but is repeated here in a slightly different form for the sake of convenience. Let F_1, F_2, F_3, F_4, F_5 , Fig. 3, be coplanar concurrent forces acting at O . If, from the end A_1 of the vector OA_1 representing the force F_1 , the vector A_1A_2 , equal and parallel to F_2 , is drawn, the vector OA_2 will represent the resultant of F_1 and F_2 . These two forces can, therefore, be replaced by the single force OA_2 . Likewise, by drawing A_2A_3 equal and parallel to F_3 , and joining O to A_3 , the resultant OA_3 of OA_2 and F_3 , or, what is the same thing, of F_1, F_2 , and F_3 , is obtained. Similarly, OA_4 is the resultant

of OA_4 and F_4 , or of F_1, F_2, F_3 , and F_4 ; and, finally, OA_5 , obtained in a similar manner, is the resultant of OA_4 and F_4 —that is, the resultant R of the five given forces. The same vector, taken in the opposite direction, as shown by the arrow Q , evidently represents the equilibrant of the same given forces.

Noting that the sides F_1, F_2, F_3, F_4, F_5 of the polygon $OA_1A_2A_3A_4A_5O$ represent the given forces in magnitude and direction, and that the closing side R or Q represents either the resultant or the equilibrant of the given forces; and noting also that for the construction of this polygon it is not necessary to draw the auxiliary vectors OA_1, OA_2 , etc., the following general rule may be stated:

Rule.—*To find the resultant or the equilibrant of any number of concurrent forces, draw, in cyclic order, vectors representing the given forces, and join the end of the last vector with the origin of the first. The vector thus obtained, if taken in cyclic order with the two vectors between which it lies, will represent the equilibrant of the given forces; if taken in non-cyclic order, it will represent the resultant.*

4. The Force Polygon.—The figure $OA_1A_2A_3A_4A_5$, Fig. 3, though not generally a closed figure, is called the **force polygon** of the given forces, and the line OA_5 is called the **closing line** of the polygon.

Usually, the lines of action of the forces are given by the center lines of the members of a structure or parts of a machine. The assemblage of these lines of action is called the **space diagram** of the given forces. It is generally more convenient to construct the force polygon separately; that is, not taking the actual meeting point of the forces as a starting point. Thus, Fig. 4 represents the force polygon (or polygons) of the forces given in Fig. 3. The point O may be taken at any convenient place, and the vectors OA_1, A_1A_2, A_2A_3 , etc., equal and parallel, respectively, to the given forces, drawn in cyclic order. The closing line A_5O represents either the resultant or the equilibrant, according to the direction in which it is taken. That this gives the

same result as the one obtained before is evident from the equality of the two polygons.

The order of succession in which the vectors are drawn is immaterial. Thus, after drawing OA_1 , A_1A_2'' may be drawn to represent F_2 , then $A_2''A_3''$ to represent F_3 , then $A_3''A_4''$ to represent F_4 , and, finally, $A_4''A_1$ to represent F_1 . The point A_2 thus determined will coincide with the one determined before, and the line A_1O will, therefore, be the same in both cases. The reason for this is obvious. If A_1A_2'' and A_1A_2 are drawn, the quadrilaterals $A_1A_2''A_3''A_4''$ and $A_1A_2A_3A_4$ will be parallelograms, since, by construction, A_1A_2'' is equal and parallel to A_1A_2 , and $A_2''A_3''$ is equal and parallel to A_2A_3 . The parallelogram $A_1A_2''A_3''A_4''$ shows that A_1A_2'' is equal and parallel to $A_1A_2 (= F_2)$; and, as in the parallelogram $A_1A_2A_3A_4$ the side A_1A_2 is equal and parallel to A_2A_3 , it follows that A_1A_2 is equal and parallel to $A_2A_3 (= F_3)$. As $A_2''A_3''$ is equal and parallel to $A_2A_3 (= F_3)$, the line drawn from A_2'' to A_3 must be equal and parallel to A_2A_3 , and, therefore, to F_3 . So that, if the point A_2 had not been already determined, it could be located by drawing from A_2'' the line $A_2''A_3$ equal and parallel to F_3 , as was stated above. It can be shown in the same manner that, whatever the order followed in drawing the sides of the force polygon, the result will always be the same.

5. The rule for the composition of concurrent forces can now be stated in the following concise manner:

Rule.—*To find the resultant or the equilibrant of several concurrent forces, draw the force polygon of the given forces, and close it; the closing line will represent either of the required forces, according to the direction in which it is taken.*

6. Conditions of Equilibrium.—Should the end of the last vector of the force polygon coincide with the origin of the first, the closing line will vanish; that is, there will be no resultant. This will indicate that the given forces are in equilibrium. Thus, if the given forces were F_1, F_2, F_3, F_4, F_5 , and Q , Fig. 3, the last vector of the force polygon would be A_5O ; the force polygon would be closed, and the forces

would have no resultant. In this case, any of the forces may be considered as the equilibrant of the others. The following principles may, therefore, be stated:

If the force polygon of a system of concurrent forces closes, the forces are in equilibrium.

Conversely, *in order that several concurrent forces may be in equilibrium, the force polygon must close.* For, otherwise, the forces would necessarily have a resultant represented by the closing line of the polygon.

COPLANAR NON-CONCURRENT FORCES

THE FORCE DIAGRAM

7. Determination of the Magnitude and Direction of the Resultant or Equilibrant of Any Number of Coplanar Non-Concurrent Forces.—Let $F_1, F_2, F_3, F_4,$

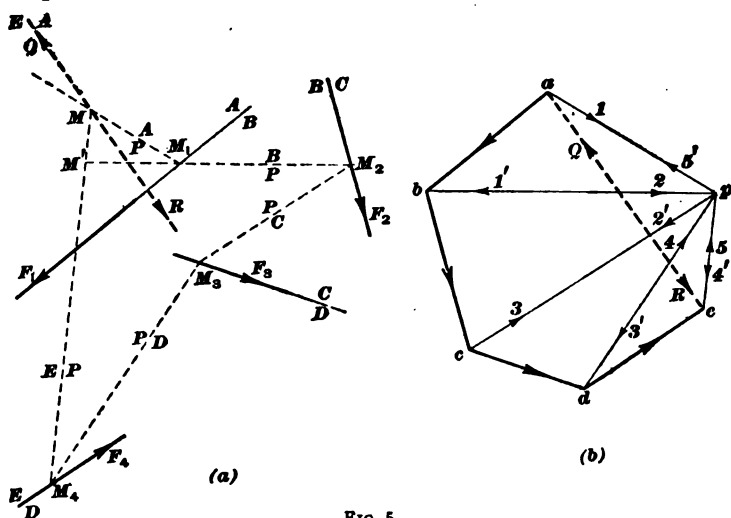


FIG. 5

Fig. 5 (a), be non-concurrent forces acting on a rigid body. It was shown in *Analytic Statics*, Part 2, that, so far as the magnitude and direction of the resultant are concerned, the

forces may be treated as concurrent; or, rather, the magnitude and direction of their resultant are the same as the magnitude and direction of the resultant of a system of concurrent forces equal and parallel to the given forces. Therefore, if a force polygon $abcde$, Fig. 5 (*b*), is constructed by drawing ab equal and parallel to F_1 , bc equal and parallel to F_2 , etc., the closing vector ae or ea will represent the magnitude and direction of either the resultant or the equilibrant of the given forces, according as it is taken in non-cyclic or in cyclic order with the two vectors ab and de between which it lies.

8. Notation.—The following notation, which is illustrated in Fig. 5 (*a*), is very convenient and very often used: The line of action of a force, instead of being indicated by letters at its extremities, is indicated by two capital letters opposite each other, one on each side of the line. For example, AB is the line of action of F_1 , and CD is the line of action of F_2 . Each letter is common to the lines of action of at least two forces, including the resultant (or the equilibrant). Thus, the lines of action of the four given forces and their resultant (the latter to be determined presently) are: AB , BC , CD , DE , and EA .

In the force polygon, shown in Fig. 5 (*b*), each vector is denoted by two small letters corresponding to the capital letters indicating the line of action of the force represented by the vector. For instance, ab represents F_1 , whose line of action is AB . Having drawn ab , the next vector bc must be drawn parallel to BC , the letter B being common to AB and BC . In the same way, after drawing bc , the next vector must be drawn parallel to the other line of action having C for one of its letters, that is, to CD . It is convenient, although not necessary, to draw the vectors of the force polygon so that the letters will be in alphabetical order. This mode of notation is known as **Bow's notation**.

If the magnitude of each force is indicated by F , with a subscript consisting of the capital letters marking the line of action of the force, we may write the following convenient

equations, showing the correspondence between the forces in the space diagram, Fig. 5 (*a*), and their magnitudes and directions as given by the force polygon, Fig. 5 (*b*):

$$F_{AB} = ab, F_{BC} = bc, F_{CD} = cd, \text{ etc.}$$

It should be noticed that the pairs of letters *AB*, *BC*, etc. indicate lines, without any reference to directions, so that *AB* has the same meaning as *BA*. The lines *ab*, *bc*, etc. of the force polygon, on the contrary, are vectors, whose directions are indicated by the order of the letters. Thus, *ab* is a vector whose arrowhead points from *a* toward *b*, and *ba* would be a vector with its arrowhead pointing from *b* toward *a*. We thus have $ab = F_{AB}$; but never $ab = ba$, the true relation between *ab* and *ba* being $ab = -ba$. The resultant of the given forces is represented by *ae*; their equilibrant, by *ea*.

9. Line of Action of the Resultant (or of the Equilibrant)—The Force Diagram.—Having determined the magnitude and direction of the resultant $R = ae$ (or of the equilibrant $Q = ea$), Fig. 5 (*b*), the line of action, or the position of this force in the space diagram, is determined in the following manner: From any convenient point *p*, Fig. 5 (*b*), draw lines *pa*, *pb*, *pc*, etc. to the vertices of the force polygon. The lines *ap* and *pb*, considered as vectors in non-cyclic order with *ab*, as indicated by the arrows 1 and 1', evidently represent the components of F_{AB} , or F_a , in the directions *ap* and *pb*; this follows from the principle of the triangle of forces. Likewise, *bp* and *pc* represent the components of F_{BC} , or F_b , in the directions *bp* and *pc*; this is indicated by the arrows 2 and 2'. The same is true of the other lines radiating from *p*. The figure formed by these lines and those of the force polygon is called a **force diagram** of the given forces. The point *p* is called the **pole** of the force diagram, and the radiating lines *pa*, *pc*, etc. are called **rays**. The position of the pole *p* being arbitrary, an infinite number of force diagrams may be drawn for the same system of forces, but they all give the same line of action for the resultant (or the equilibrant).

10. To determine this line of action, draw from any point M_1 , Fig. 5 (a), on the line of action AB of F_1 , two lines PA and PB , parallel, respectively, to the components 1 and 1' of F_{AB} , represented in the force diagram by the vectors ap and pb , respectively. Notice that the lines PA and PB are denoted by capital letters corresponding to the small letters denoting the components 1 and 1'. Produce PB to its intersection with BC at M_2 (the line BC being that line in the space diagram having one of its letters, B , the same as one of the letters of the line PB just drawn). From M_2 draw PC parallel to pc , or 2', and produce it to its intersection M_3 with CD , the latter being that line in the space diagram one of whose letters, C , is the same as one of the letters of the line PC just drawn. From M_3 draw PD parallel to pd , meeting DE at M_4 . Finally, from M_4 draw PE parallel to pe , meeting at M the line PA drawn from M_1 . The line EA , drawn through M parallel to ae , is the line of action of either the resultant R or the equilibrant Q .

The correctness of this construction is very easily demonstrated. Each of the forces F_{AB} , F_{BC} , etc. can be replaced by its two components, as given in the force diagram. The force F_{AB} may be replaced by two components, equal, respectively, to ap and pb , acting along the lines AP and PB , and applied anywhere on those lines. These components will be designated by F_{AP} and F_{PB} . The force F_{BC} may be replaced by its two components F_{BP} and F_{PC} , equal and parallel to bp and pc , respectively, and acting along the lines BP and PC . Similarly, F_{CD} may be replaced by the components F_{CP} and F_{PD} , and F_{DE} by the components F_{DP} and F_{PE} . It should be noted, now, that the component F_{PB} of F_{AB} is equal and opposite to the component F_{BP} of F_{BC} , as is plainly shown by the opposite vectors pb and bp . Likewise, the component F_{PC} of F_{BC} is equal and opposite to the component F_{CP} of F_{CD} , and the component F_{PD} of F_{CD} is equal and opposite to the component F_{DP} of F_{DE} . These components, therefore, balance in pairs, and may be removed. This leaves the system reduced to the two forces F_{AP} and F_{EP} , whose resultant (or equilibrant) must be the same as the resultant (or

equilibrant) of the given forces. The line of action of that resultant (or equilibrant) must, therefore, pass through M , the point of intersection of its two components. In the force diagram, the two components of the equilibrant Q are represented by the vectors pa and pe , whose directions are indicated by the arrows s' and s .

THE FUNICULAR OR EQUILIBRIUM POLYGON

11. Definition and General Properties of the Funicular.—The polygon $MM_1M_2M_3M_4$, Fig. 5 (*a*), whose vertices are on the lines of action of the forces (the resultant or the equilibrant included), and whose sides are parallel to the rays of the force diagram, is called an **equilibrium polygon**, a **funicular polygon**, or simply a **funicular** of the given forces. Since both the pole p of the force diagram, Fig. 5 (*b*), and the starting point M_1 , Fig. 5 (*a*), of the funicular are arbitrary, it is evident that to any system of forces there corresponds an infinite number of funiculars. The vertex M of the funicular, however, is always on the line of action of the resultant R or the equilibrant Q of the given forces.

12. The sides of the funicular are called **strings**. As already explained, they are the lines of action of the components represented in magnitude and direction by the rays of the force diagram.

In the space diagram, *the two strings parallel to the rays representing the components of one of the forces intersect on the line of action of that force. Each string is common to two forces whose lines of action have a common letter. Also, each string is drawn between the lines of action of the two forces whose vector representatives in the force polygon intersect on the ray parallel to the string in question.* Thus, the string PB is parallel to the ray pb , and is drawn between AB and BC , whose corresponding vectors ab and bc meet on the ray pb .

These are convenient relations to remember, as they facilitate the work of construction, making it to some extent mechanical.

13. There is another important and useful property of the funicular, namely: *If any two strings of the funicular are produced, their point of intersection will be a point on the line of action of the resultant (or the equilibrant) of the forces acting through the other vertices of the new funicular thus formed.*

For example, if PB , Fig. 5 (*a*), is produced to its intersection M' with PE , a new funicular $M_1M_2M_3M'$ will be formed, and M' will be a point in the line of action of either the resultant or the equilibrant of the forces F_2, F_3, F_4 , acting through the vertices M_2, M_3, M_4 . This follows from the general principles explained in connection with the determination of the point M . When the force F_1 is left out of consideration and the funicular is constructed, beginning at M_1 , PB is its first string, and the point M' where this string intersects the last one PE determines a point in the line of action of either the resultant or the equilibrant of the forces considered. The magnitude and direction of the resultant, are given by the vector be (not drawn) in the force polygon, Fig. 5 (*b*).

14. The Funicular as a Jointed Frame.—Suppose the strings PA, PB, PC, PD, PE , Fig. 5 (*a*), to be replaced by bars jointed at the points M_1, M_2 , etc., and suppose also the forces F_1, F_2, F_3, F_4 and the equilibrant Q to act at these joints, respectively. It is evident, then, that the frame formed by the bars will be in equilibrium, for each force can be replaced by two components—one along each of the bars at the intersection of which the force acts. But it has been explained that, for either component of any of the forces, there is an equal and opposite component of another force, balancing the former component. For example, the component of F_2 along M_1M_2 is equal and opposite to the component of F_1 along the same line. Each bar being thus in equilibrium, the whole frame is in equilibrium.

It is also obvious that the equal and opposite components acting along any of the bars measure the stress in that bar. The following proposition may, therefore, be stated:

When a frame with no diagonal members is in equilibrium under the action of forces applied at its joints, the frame is a funicular of the applied forces, and the rays of the force diagram represent the magnitudes of the stresses in the members of the frame to which they are parallel.

15. Special Conditions.—In the case represented in Fig. 5 (b), the origin a of the first vector and the end e of the last vector of the force polygon do not coincide. In this case, the forces have a single resultant R represented in magnitude and direction by the vector ac , and whose line of action AE , Fig. 5 (a), is determined by constructing

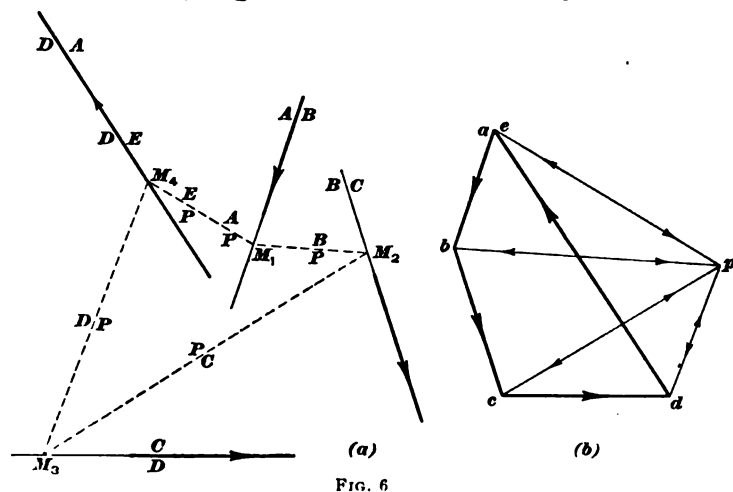


FIG. 6

the funicular as explained above. Since PA and PE are parallel, respectively, to the two intersecting lines pa and pc , they must intersect and thus determine a point M in the line of action of the resultant (or equilibrant).

If the points a and c of the force polygon coincide; that is, if the force polygon closes, the forces either form a balanced system or are equivalent to a couple. Which of these two conditions obtains is determined by means of the funicular as follows:

1. In Fig. 6 is represented a system of four forces, F_{AB} , F_{BC} , F_{CD} , and F_{DE} , whose force polygon $abcde$ closes; that

is, when the vectors ab , bc , cd , and de are drawn to represent the given forces, the end e of the last vector de falls on the origin a of the first. Taking any pole p , and drawing the rays pa , pb , pc , and pd , the ray pe coincides with pa . Starting from any point M_1 on AB , PA and PB are drawn parallel, respectively, to pa and pb . From M_2 , where PB intersects BC , PC is drawn parallel to pc , meeting CD at M_3 . From M_3 , PD is drawn parallel to pd , meeting PA at M_4 . Finally, from M_4 , PE is drawn parallel to pe . As pa coincides with pe , it is evident that PA and PE must either

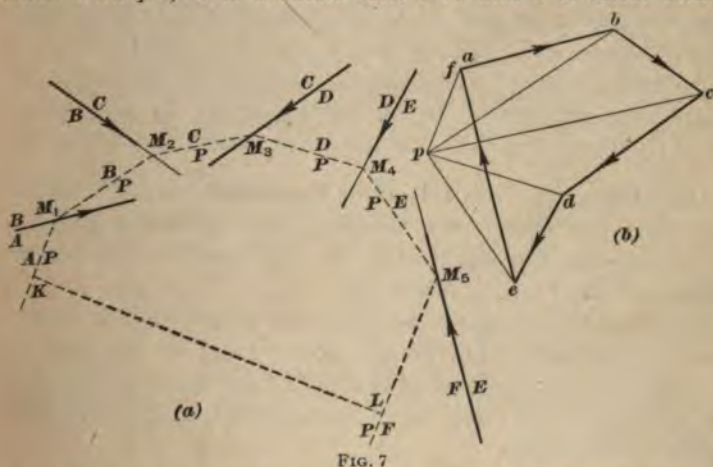


FIG. 7

coincide or be parallel. In the case represented in the figure, they coincide; this shows that the given forces form a balanced system. For, as already stated, the sides of the funicular are lines of action of components represented in magnitude and direction by the rays of the force diagram. In the case considered in Fig. 5, all these components balanced in pairs, except the extreme components ap and pe ; and this left the system reduced to two forces acting along PA and PE , whose resultant was the resultant of the given forces. In the present case, the components bp and pb , cp and pc , dp and pd form balanced pairs, as before; and as the remaining components ap and pe are equal in magnitude and opposite in direction, and have the same line of action PA or PE ,

they balance each other. The system is, therefore, in equilibrium.

2. In Fig. 7 are represented five forces F_{AB} , F_{BC} , F_{CD} , F_{DE} , and F_{EF} , whose force polygon $abcdef$ closes, the end of the vector ef coinciding with the origin of the vector ab . As before, the force diagram is completed by drawing the rays from any pole p , and then the funicular $M_1 M_2 M_3$, etc. is constructed. The given forces can be replaced by two forces represented by the vectors pa and $fp (= ap)$. Here, however, these components, although equal and opposite, have not the same line of action, but act along the parallel lines PA and PF . The system is, therefore, equivalent to a couple whose moment is obtained by multiplying the magnitude of the force represented by the vector pa by the perpendicular KL between the sides of the funicular parallel to that vector.

16. Graphic Conditions of Equilibrium.—From the foregoing discussion, the necessary and sufficient graphic conditions for the equilibrium of any system of coplanar forces follow at once. They are:

1. *The force polygon must close;* for, otherwise, the vector drawn from the origin of the first vector to the end of the last (that is, the closing line of the polygon) would give the magnitude and direction of the resultant, which obviously would not be zero.

2. *The last string PE , Fig. 6, of the funicular must coincide with the first PA ;* for, otherwise, the forces would be equivalent to a couple, as explained in the last article.

Conversely, *if the force polygon of a system of forces closes, and the last string of the funicular coincides with the first, the forces form a balanced system.*

NOTE.—When the funicular is spoken of, any funicular that is drawn is meant.

17. Parallel Forces.—When the given forces are all parallel, the construction is much simplified, as in such a case the force polygon reduces to a straight line. In Fig. 8, F_{AB} , F_{BC} , F_{CD} , F_{DE} , and F_{EF} are given parallel forces acting along the lines AB , BC , etc. in the directions shown by the

arrows. It should be noted that letters which are common to the lines of action of two forces are written only once and placed between the two lines to which they are common. This is often done when there is no danger of confusion; otherwise it is preferable to repeat the letters, as in the funicular $M_1 M_2 M_3$, etc., shown in the same figure. Were there no lines crossing the polygon, it would be sufficient to write the letter P once anywhere inside of it. In the present case, however, this might be confusing.

To construct the force diagram, draw an indefinite line XY , Fig. 8 (b), parallel to the common direction of the forces. From any convenient point, as a , lay off the vector ab to represent F_{AB} (no arrowhead is necessary, as the direction of the vector is indicated by the order of the letters a and b).

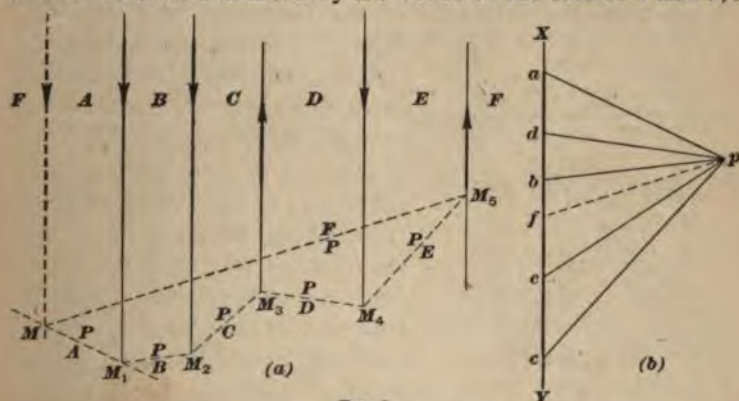


FIG. 8

From b lay off the vector bc to represent F_{BC} ; from c lay off the vector cd to represent F_{CD} ; and so on for the other forces. The last vector is ef , representing F_{EF} . The closing line of the polygon is the line af joining the origin of the first vector with the end of the last. The resultant of the given forces is, therefore, represented in magnitude and direction by af ; the equilibrant, by fa .

The line of action AF of the resultant (or the equilibrant) is found in the usual manner, by selecting a pole p , drawing the rays pa , pb , etc., and then constructing the funicular $M_1 M_2 M_3 M_4 M_5 M_6$.

GENERAL SOLUTION OF SOME IMPORTANT PROBLEMS

PROBLEMS ON CONCURRENT FORCES

18. Problem I.—*To resolve a force into two components along lines meeting on the line of action of the force.*

This problem has been of constant occurrence in *Analytic Statics*, and its graphic solution has been indicated as a necessary step in the analytic solution. It is repeated here as a review of the principle of the parallelogram of forces

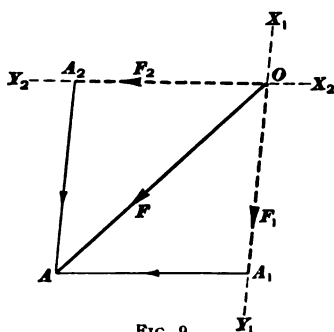


FIG. 9

and the derived principle of the triangle of forces, which principles form the foundation of all graphic methods.

Let F , Fig. 9, be a force acting at O , and let X_1Y_1 and X_2Y_2 be lines along which it is desired to resolve F . The resolution may be accomplished by drawing from A the line AA_1 parallel to Y_2X_2 , meeting X_1Y_1 at A_1 ; or AA_2 parallel to Y_1X_1 , meeting X_2Y_2 at A_2 . In the former case, the component $OA_1 = F_1$ along X_1Y_1 is determined in magnitude, position, and direction, and the component $OA_2 = F_2$ in magnitude and direction, its position being understood to be OA_2 along X_2Y_2 ; in the latter case, $OA_2 = F_2$ is determined in magnitude, position, and direction, and $OA_1 = F_1$ in magnitude and direction, its position being understood to be OA_1 along X_1Y_1 .

Usually, it is not necessary to show the components in position, and then either of the triangles OA_1A_2 , OA_2A_1 , is sufficient for the solution of the problem. Should it be desired to show the component in position, it is better to

construct the two triangles, or the parallelogram OA_1AA_2 . The components can also be determined by means of one triangle, as OAA_1 , and then $OA_2 = A_1A$ may be laid off along OY_2 without drawing AA_2 .

The last remark, be it understood, refers to the case in which the components are to be shown in position in the space diagram. Usually, however, as has been repeatedly observed, it is neither necessary nor convenient to show the magnitudes of all the forces in the space diagram, and the geometric constructions are better effected separately. This is almost invariably done in the graphic solution of static problems, and the principle of the triangle of forces, from which the polygon of forces is directly derived, is used.

19. The general method used in graphic statics for the solution of the problem stated and solved above is as follows:

Let the given force be F_{AB} , Fig. 10, and let the lines of action of the required components be BC and CA . At any convenient place draw a vector ab to represent F_{AB} .

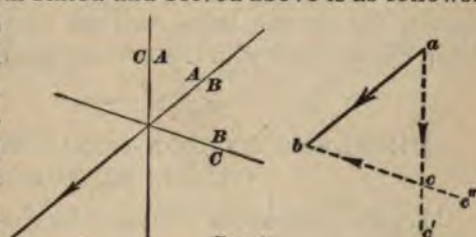


FIG. 10

From a and b draw the indefinite lines ac' and bc'' , parallel, respectively, to AC and BC , and meeting at c . The vectors ac and cb , both in non-cyclic order with ab , are the required components of F_{AB} .

20. Problem II.—*In a balanced system of concurrent forces, the lines of action, magnitudes, and directions of all the forces but two are known, and also the lines of action of the other two. It is required to find the magnitudes and directions of these two.*

Let the forces be six, of which F_{AB} , F_{BC} , F_{CD} , F_{DE} , Fig. 11 (a), are completely known, while of the other two, only the lines of action EF and AF are known. Starting at any convenient point a , Fig. 11 (b), draw the force polygon $abcde$ for the known forces. Through a draw the indefinite

line $a'f'$, parallel to AF of Fig. 11 (a); and through e draw the indefinite line $e'f''$, parallel to EF of Fig. 11 (a), and intersecting $a'f'$ at f . Then will ef and fa be the required

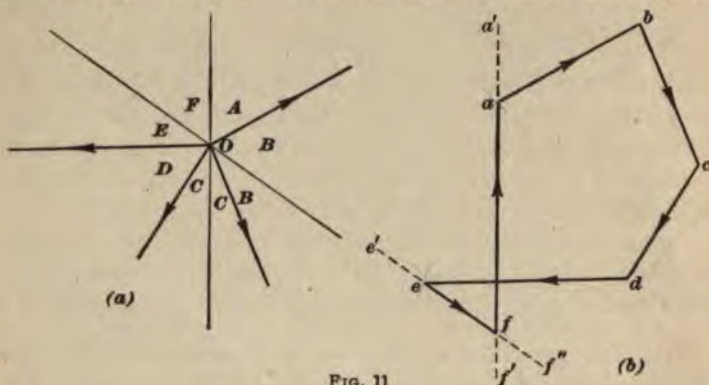


FIG. 11

forces, the arrows being marked so that each vector will be in cyclic order with the two between which it lies.

PROBLEMS ON NON-CONCURRENT AND ON PARALLEL FORCES

21. Problem III.—*A rigid body is in equilibrium under the action of several forces of which all but two are wholly known. Of the other two, the line of action of one and a point in the line of action of the other are known. It is required to determine completely the two latter forces.*

Let the forces be six, of which F_{AB} , F_{BC} , F_{CD} , and F_{DE} are entirely known, as shown in Fig. 12 (a). Of the fifth force F_{EF} , only the line of action EF is known; and of the sixth force F_{FA} , only a point M_a on its line of action is known.

Draw the force polygon $abcde$, Fig. 12 (b), for the known forces; and, taking any pole p , draw the rays pa , pb , pc , pd , and pe . Through e draw the indefinite line $e'f'$ parallel to the given line of action EF . Construct the funicular for the given forces, drawing the first string PA so that it will pass through M_a , which is done by drawing PA through M_a and parallel to pa , to meet AB at M_1 , and then drawing PB ,

PC , etc. in the usual manner. The string PE , from the last of the known forces DE , meets the given line of action of EF at M_5 . Draw M_5M_4 , and from the pole p draw pf parallel to M_5M_4 , meeting $e'f'$ at f . Finally, draw fa ; then will the vector ef represent F_{EF} and the vector fa will represent

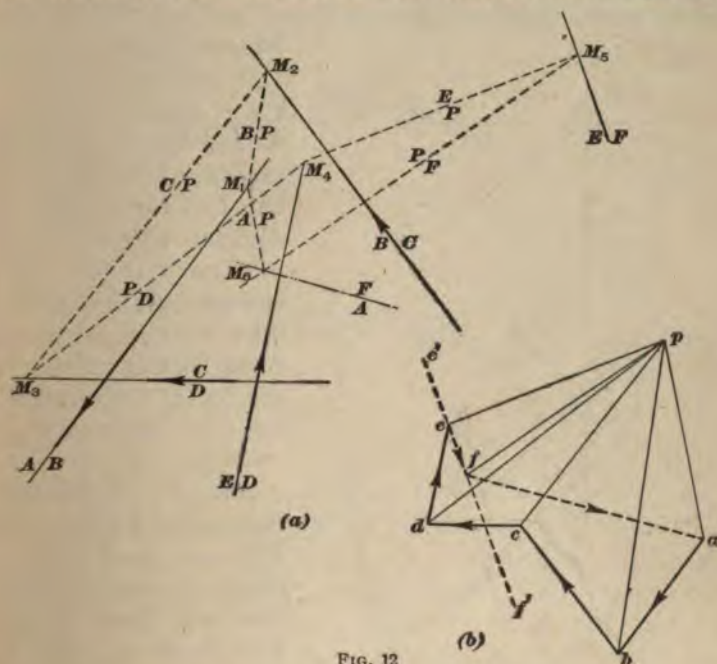


FIG. 12

F_{FA} , the required force through M_4 . The line of action of the latter force is FA , drawn through M_4 parallel to fa .

22. Special Case of the Preceding Problem: Four Forces.—An important special case of the preceding problem is that in which the number of forces is four, and the lines of action of the two completely known forces intersect. The solution may be effected either by the general method explained above, or by the following special method:

Let F_{AB} and F_{BC} , Fig. 13, be the wholly known forces; CD , the line of action of one of the partly unknown forces; and K , a point in the line of action of the other.

First determine the resultant R of F_{AB} and F_{BC} , by the force triangle abc . The line JH , drawn parallel to ac through the intersection H of AB and BC , is obviously the line of action of R . It is also obvious that R is the equilibrant of the yet unknown forces F_{CD} and F_{AD} . These three forces must, therefore, be concurrent, so that, if I is the

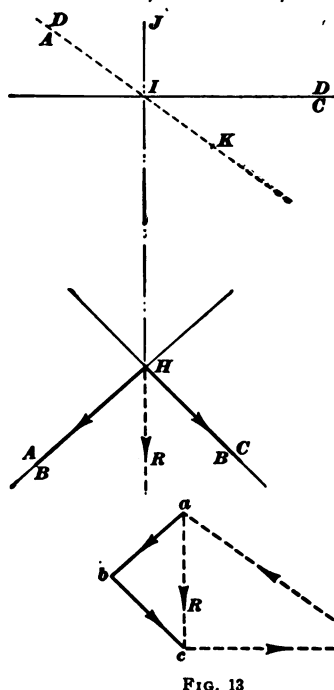


FIG. 13

intersection of HJ and CD , the line of action of F_{DA} is determined by joining K to I . Having the lines CD and DA , the force polygon is completed by drawing cd'' parallel to CD , and ad' parallel to AD . The intersection of these two parallels is the fourth vertex of the force polygon.

23. Problem IV:

Polygonal Frame.—A polygonal frame (that is, a frame without diagonal members) is in equilibrium under the action of forces acting at the joints. Of these forces, one is

wholly known, and the lines of action of all but one of the others are known. It is required to determine the unknown elements of the forces not wholly known, and also the magnitudes and characters of the stresses in the members of the frame.

Let J_1, J_2, J_3, J_4, J_5 , Fig. 14, be the given frame, F_{AB} , the force that is wholly known, and BC, CD, DE, EF the lines of action of the forces acting at J_2, J_3 , etc. The force F_{FA} acting at J_1 is entirely unknown. The letters B, C, D , it will be noted, are written only once, each in the space bounded by the lines to which the letter is common; the

letters A and E are repeated for convenience. The letter P , being common to all the sides of the frame (since these sides, as explained in Art. 14, form the strings of the funicular of the forces F_{AB} , F_{BC} , etc.), is written only once, in the center of the polygon.

The forces F_{AB} , F_{BC} , etc. are external forces, which induce

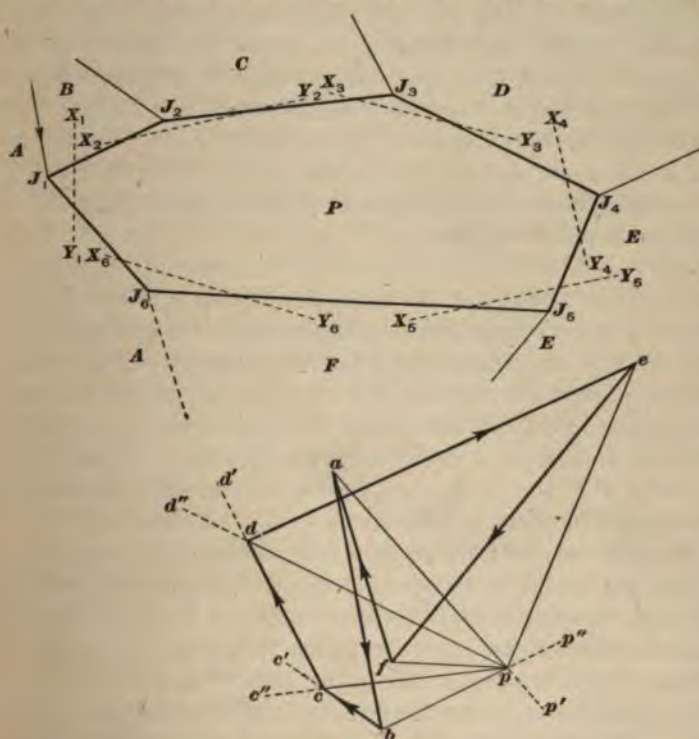


FIG. 14

in the bars internal forces. The latter occur in pairs, each pair consisting of the equal and opposite forces acting along a bar. Either of these forces measures the stress in the bar.

In the general problems solved and discussed in previous articles, the funicular has not been given, and the pole of the force diagram has been assumed arbitrarily. Here, however, the pole is determined by the data. Having drawn the

vector $ab = F_{AB}$, the rays from a and b must be parallel to the strings PA and PB , respectively—that is, to $J_1 J_2$ and $J_1 J_3$. Therefore, if from a and b are drawn ap' and bp' parallel, respectively, to AP and BP , their intersection p will be the pole. The next line in the force polygon must be parallel to BC , and the next ray must be parallel to PC . Therefore, if bc' and pc'' , parallel, respectively, to BC and PC , are drawn, their intersection c will be the end of the vector bc , representing F_{BC} . Likewise, the intersection d of the lines cd' and pd'' , parallel, respectively, to CD and PD , is the end of the vector cd representing F_{CD} . The forces F_{DE} and F_{EF} are determined in the same manner. Finally, the closing vector fa of the force polygon determines F_{FA} in magnitude and direction.

24. The stresses in the members are determined by the rays of the force diagram. If, for instance, the two members PB and PA meeting at the joint J_1 are cut by a plane $X_1 Y_1$, and the part at the right of $X_1 Y_1$ is removed, the part on the left will be in equilibrium under the action of the force F_{AB} , and two forces (now considered as external) acting along PA and PB ; the latter two forces measure the stresses in those two members. (See *Analytic Statics*, Part 1.)

Now, in the force diagram, ab represents F_{AB} , and, since bp and pa are parallel, respectively, to PB and PA , or $J_1 J_2$ and $J_1 J_3$, the vectors bp and pa , taken, as indicated by the order of the letters, in cyclic order with ab , will represent the equilibrants of F_{AB} along the lines $J_1 J_2$ and $J_1 J_3$, and will, therefore, measure the stresses in PB and PA , respectively. These stresses will be designated by the notation S_{PB} , S_{PA} . Their characters are easily determined by the directions of the vectors. Thus, when the portion of the frame at the right of $X_1 Y_1$ is removed and the forces bp and pa are applied, respectively, at the intersections of $X_1 Y_1$ with $J_1 J_2$ and $J_1 J_3$, it is seen that bp , whose direction is from J_1 to J_2 , acts away from the joint, showing that the stress in PB is a pull. The force pa , on the contrary, will act toward the joint J_1 , which shows that the stress in PA is a thrust.

The stresses in the other members are determined in a similar manner.

25. Stress Diagram.—It will be noticed that, for the determination of the stresses, the triangle formed by each vector of the force polygon and the rays drawn to its extremities is considered as a force triangle. If there were more than one external force acting at any joint, the polygon formed by the vectors representing these forces and by the rays parallel to the members meeting at the joint would be used as a secondary force polygon, the same as the force triangle has been used above.

When the force diagram is employed as a combination of force polygons for the determination of stresses, it is called a **stress diagram**. If each joint of the frame is denoted by the letters indicating the members meeting at the joint, the triangle having corresponding letters in the stress diagram will be the force triangle for that joint. Thus, the joint J_1 may be called joint DEP , and the triangle dep in the stress diagram is the force triangle for the equilibrium of the forces acting at J_1 , from which triangle the stresses S_{PD} and S_{PE} are determined.

Arrowheads might be drawn on the rays of the stress diagram to indicate the direction in which forces represented by those rays are to be applied to the members cut; but, as in this case each ray would have two arrows pointing in opposite directions, these arrows might prove a source of confusion. Besides, the arrowheads of the force polygon are sufficient for the determination of the directions of the forces represented by the rays. Consider, for example, the joint J_1 , or ABP , whose corresponding force triangle is abp . Since, in this triangle, the vectors must be in cyclic order, and since the origin and the end of the vector ab are known, the direction of each of the other two vectors follows at once. The end b of ab must be the origin of the next vector, which is, therefore, bp and not pb . Likewise, the other vector is pa and not ap .

ILLUSTRATIVE EXAMPLES

26. Triangular Truss.—A triangular frame or truss $J_1 J_2 J_3$, Fig. 15, supported at the joints J_1 and J_3 , which are on the same horizontal line, carries a weight of 800 pounds at the upper joint J_2 . The dimensions being as shown, it is required to determine the reactions of the supports, and also the stresses in the three members of the truss.

Owing to the symmetry of the structure, the reactions, supposed to be vertical, are each equal to one-half the load, or 400 pounds. Their lines of action are indicated by AB and BC . The line of action of the applied weight is the

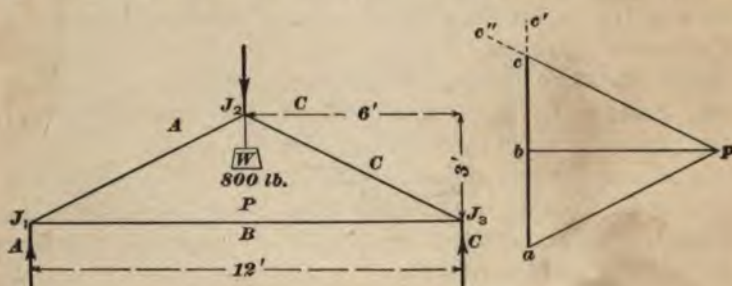


FIG. 15

vertical AC , through J_2 . For convenience, the magnitude and direction of this weight are represented by a vector with its arrowhead pointing toward J_2 from above. The notation does not require any explanation, as it is the same notation used in previous articles.

As explained in Art. 14, the truss $J_1 J_2 J_3$ is a funicular of the forces F_{AB} , F_{BC} , F_{AC} ; and, as explained in Art. 25, the stress diagram is a combination of force triangles, each of which consists of vectors representing the forces acting at a joint. Starting with the joint J_1 , the vector $a b = F_{AB}$, and the rays ap and pb , parallel, respectively, to the members AP and PB , are drawn. These rays meet at the pole p . Then bp and pa will represent S_{BP} and S_{PA} , respectively. Since bp , when applied along BP , points away from the joint J_1 , the stress S_{BP} is a pull. The force pa , on the contrary, when

applied along PA , is directed toward the joint, which shows that S_{PA} is a thrust.

The line pc'' may be drawn parallel to PC to meet bc' (parallel to BC , and, in this case, in line with ab) at c , and the triangle bcp will give both F_{BC} and the stress in PC ; but this is not necessary, as, owing to the symmetry of the truss, $F_{AB} = F_{BC}$, and $S_{PA} = S_{PC}$.

27. Scale Used.—In the solution of this problem, a scale of 1 inch to the foot was used for distances, making the drawing of the truss 12 inches long and 3 inches high. For the forces, a scale of 100 pounds to the inch was used, making ab 4 inches long. If all the stress diagram acp had been drawn, this scale might have been inconveniently large; but only the diagram abp for the joint ABP or J_1 was drawn, this being sufficient, as already explained. For this kind of work, a decimally graduated triangular scale (one having scales divided into tenths, twentieths, thirtieths, etc. of an inch) is preferable to the scales used in ordinary mechanical drawing. The scale of fortieths is a convenient one to use. In the present case, $\frac{1}{40}$ inch represented 2.5 pounds, and assuming that the drawing was correct within $\frac{1}{40}$ inch, the forces determined by it may be considered correct within 2 or 3 pounds. The line bcp was found by actual measurement to be 8 inches, and the line pa , $8\frac{37}{40}$ inches. Denoting, therefore, compression and tension by the plus and the minus sign, respectively,

$$\begin{aligned} S_{BP} &= -8 \times 100 = -800 \text{ pounds,} \\ S_{AP} &= S_{CP} = +8\frac{37}{40} \times 100 \\ &= +(800 + 37 \times 2.5) = +893 \text{ pounds.} \end{aligned}$$

No fractions of a pound are written, as the results cannot be expected to be exact within less than 2 or 3 pounds.

In the solution of static problems by graphic methods, as large a scale as possible should be employed. The draftsman should use his judgment as to the degree of approximation attained. If, for instance, he thinks that, on an average, the lengths of the lines are correct within $\frac{1}{80}$ inch, he may measure them in inches and fiftieths, and consider the forces, obtained

in this manner, as correct within about as many pounds or tons as are represented by $\frac{1}{80}$ inch, according to the scale used. Thus, if the scale used is 1 ton to the inch, a length of $\frac{1}{80}$ inch will represent $\frac{1}{80}$ ton = $\frac{2000}{80}$ pounds = 40 pounds; and a line found to measure $4\frac{7}{80}$ inches in length will represent $4 \times 2,000 + 7 \times 40 = 8,280$ pounds, within *about* 40 pounds.

28. Roof Truss.—A roof truss $J_1 J_2 J_3$, Fig. 16, with dimensions as shown, is loaded at the joints in the manner indicated in the figure. The supports J_1 and J_3 are on the same level, the member $G P_3$, or $J_2 J_3$, is horizontal, and the members $P_1 P_2$ and $P_2 P_3$ are perpendicular, respectively, to $J_1 J_2$ and $J_2 J_3$ at their middle points. The forces acting at the joints are supposed to be the resultants of weights, and act, therefore, vertically. The reactions F_{AG} and F_{GF} are also supposed to be vertical. It is required to determine the magnitudes of these reactions and the magnitudes and characters of the stresses in the members of the truss.

From the symmetric distribution of the forces, each reaction is equal to one-half their sum; that is, $F_{AG} = F_{GF} = 2,000$ pounds. It will be observed that each triangle in the truss is designated by a single letter, common to the lines of the triangle. Also, the lines $J_1 J_2$, $J_2 J_3$, $J_1 J_3$ have the common letter G . The reason for this is that in the force diagram the rays parallel to the three sides of each triangle radiate from a common point, and that the rays parallel to $J_1 J_2$, $J_2 J_3$, and $J_1 J_3$ also radiate from a common point, as will be seen presently.

For the determination of the stresses, any joint may be taken at random, and the force polygon for the forces acting at that joint constructed, provided that all the forces but two are known. Usually, one of the joints at the supports is the most convenient to begin with, not only because the stress diagram is thus more methodically constructed, but also because, in the majority of cases, the joints at the supports are the only ones for which the force polygon can be constructed independently of the force polygons corresponding to the other joints.

Starting with J_1 , there are four forces acting at that joint; namely, F_{GA} , F_{AB} , S_{BP_1} , and S_{P_1G} . Of these, the first two are known; therefore, the other two may be determined by constructing the force polygon $ga b p_1 g$, as in the problem solved in Art. 22; that is, by drawing, to any convenient scale, $ga = F_{GA} = 2,000$ pounds; $ab = F_{AB} = 500$ pounds, and then from b a parallel bp_1' to BP_1 , and from g a parallel

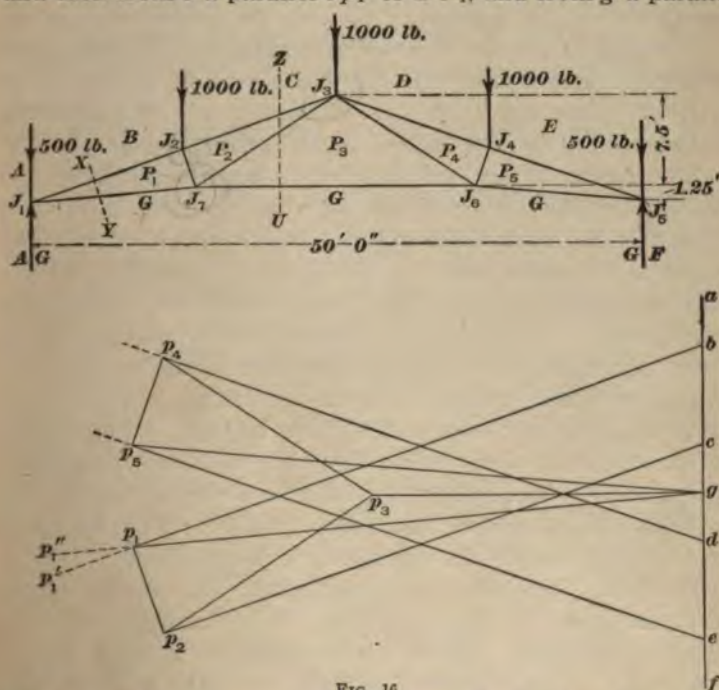


FIG. 16

gp_1'' to GP_1 , meeting bp_1' at p_1 . As GA and AB are parallel (or, in this case, coincident), the vectors ga and ab run along the same line, but in opposite directions. The lengths of the lines bp_1' and p_1g , multiplied by the scale of reduction, give S_{BP_1} and S_{P_1G} . If bp_1' were applied along BP_1 , its direction would be toward the joint J_1 ; and, if p_1g were applied along P_1G , its direction would be away from joint J_1 . Hence, S_{BP_1} is a thrust and S_{P_1G} a pull; that is, BP_1 is in compression, and P_1G is in tension. It is to be repeated

here that, in determining S_{BP_1} and S_{P_1G} , what has been really determined in either case is one of the two equal and opposite forces constituting the stress in the member in question; the force, namely, consisting in the action of the member on the joint considered. Thus, the vector $b p_1$ represents the action of the member $B P_1$ on the joint J_1 . As stated in various other places, the members $B P_1$ and $P_1 G$ may be imagined cut by a plane XY , and the force polygon, as constructed, gives the external forces $b p_1$ and $p_1 g$ that must be applied at the intersections of XY with $J_1 J_2$ and $J_1 J_3$ in order to balance ga and ab . These forces represent the action exerted by the part of the truss at the right of XY on the part at the left, which action is transmitted along $J_1 J_2$ and $J_1 J_3$. The action of the part of the truss at the left of XY on the part at the right is equal and opposite to the action before referred to; so that, if the equilibrium of the part on the right is considered, the external forces that must be applied to the members cut must be equal and opposite to those introduced when the equilibrium of the other part was considered; and, therefore, in the construction of the force polygon, the vectors $b p_1$ and $p_1 g$ may be used, but with their directions reversed, in which case they become $p_1 b$ and $g p_1$, respectively. The same thing may be expressed by saying that the actions of a member on the joints at its extremities are equal in magnitude, but opposite in direction.

The joint J_1 is to be considered next. The forces acting on it are $F_{P_1 B}$, F_{BC} , F_{CP_1} , and $F_{P_1 P_2}$, of which only the last two are unknown. In the stress diagram already drawn, $p_1 b = S_{P_1 B}$. Laying off $bc = F_{BC} = 1,000$ pounds, and drawing from p_1 and c parallels to $P_1 P_1$ and CP_1 , respectively, $p_1 b c p_1$ is determined as the force polygon for the joint J_1 ; this determines S_{CP_1} and $S_{P_1 P_2}$.

Now pass to J_2 , for which S_{GF_2} and $S_{F_2 P_2}$ are known and represented in the stress diagram by $g p_2$ and $p_2 p_3$ (notice the order of the letters). Drawing $p_2 p_3$ and $g p_2$, parallel, respectively, to $P_2 P_2$ and $G P_2$, $g p_2 p_3 g$ is determined as the force polygon for the joint J_2 , from which $S_{P_2 P_3}$ and S_{GF_2} can be determined.

Since the truss is symmetrical, it is not strictly necessary to draw the rest of the stress diagram. When space permits, however, it is advisable to complete the diagram, as this affords a good check on the work. For joint J_2 , one force, F_{CD} , is given, and two, $S_{P_3P_2}$ and S_{P_2C} , have already been determined. These forces are represented by cd , p_2p_3 , and p_2c . The lines dp_4 and p_3p_4 , parallel, respectively, to DP_4 and P_3P_4 , complete the polygon $p_3p_2cdp_4p_3$ for the joint J_2 , or $P_3P_2CDP_4P_3$. If the work has been accurately done, there must obtain $dp_4 = cp_3$, $p_4p_3 = p_2p_3$. The construction of the polygon for the joints J_4 , J_3 , and J_6 is left as an exercise for the student.

It will be again noted that the force polygon for any joint has the same letters as the lines meeting at the joint, and that the directions of its vectors are obtained by starting with one whose direction is known and taking the others in cyclic order. Thus, for the joint J_4 , or DEP_4P_4D , the polygon is dep_4p_4d . The direction of ep_4 shows that EP_4 is in compression, etc. In the case of J_7 , where there is no external force applied, it will be observed that the direction of the vector parallel to P_1G must be opposite what it is for the joint J_1 . Now, for this joint, the force polygon is $gabp_1g$, in which the vector parallel to P_1G is p_1g . Therefore, the direction of the vector represented by the same line in the polygon for J_7 must be gp_1 , and the direction of the other vectors is determined as above.

By actual measurement, the following results, which the student should verify, have been found:

$$S_{BP_1} = S_{EP_5} = + 6,150 \text{ pounds}$$

$$S_{CP_2} = S_{DP_4} = + 5,850 \text{ pounds}$$

$$S_{GP_1} = S_{GP_5} = - 5,850 \text{ pounds}$$

$$S_{P_1P_2} = S_{P_5P_4} = + 950 \text{ pounds}$$

$$S_{P_2P_3} = S_{P_3P_4} = - 2,600 \text{ pounds}$$

$$S_{GP_3} = - 3,350 \text{ pounds}$$

These results are correct within about 30 pounds. Besides the construction of the diagrams, the work should be checked by the method of sections (see *Analytic Statics*, Part 2). If the truss is cut by the plane ZU , intersecting the three

members CP_3 , P_2P_3 , and P_3G , and moments are taken about J_3 , we get, remembering that $F_{GA} = 2,000$ pounds,

$$2,000 \times 25 - 500 \times 25 - 1,000 \times 12.5 - S_{GP_3} \times 7.5 = 0$$

whence,
$$S_{GP_3} = \frac{25,000}{7.5} = 3,333 \text{ pounds}$$

which differs but little from the result found graphically and given above.

29. The vectors representing the external forces have been drawn as they have been required. Usually, however, it is better to begin by constructing the complete force polygon (in this case a straight line) for the external forces. This procedure is strictly necessary when the reactions are to be determined graphically.

30. Crane.—A weight of 12 tons is supported by a crane

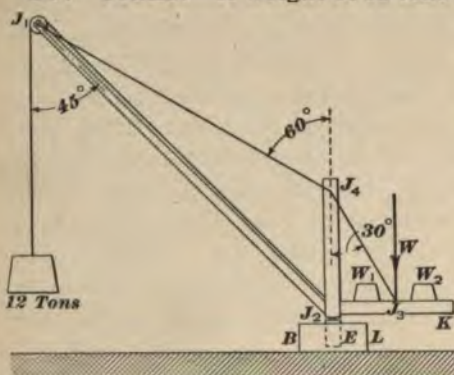


FIG. 17

consisting of a mast or post EJ_1 , Fig. 17, that fits and may turn in a socket in a block BL ; an inclined post or jib J_2J_1 , from which the weight is suspended; a stay J_1J_4 , which may be either a rod or a rope, and the backstay J_4J_2 , which also may be either a rod or a

rope, and is fastened to a horizontal plank J_2K . The inclinations of the various members to the vertical are shown in the figure. In order to avoid bending and upsetting, it is desirable that the resultant of the external forces applied at the crane should be vertical and pass through the support E of the post. To accomplish this end, weights W_1 , W_2 are placed on J_2K to counteract the upsetting effect of the eccentric load acting through J_1 . These weights are supposed to be equal and placed at equal distances from J_2 , so that their resultant W may be treated as a single weight acting

through J_2 . It is required to find the weight W , the reaction at E , and the stresses in the various members.

As usual, external forces are assumed to act at the intersections of the center lines of the members directly affected by them, and internal forces along those center lines. Fig. 18 shows a skeleton diagram of the crane, and a force diagram whereby the weight $W = F_{CA}$, the reaction F_{BC} at J_2 , and the stresses in the members are determined. The notation need not be explained. To determine F_{BC} and F_{CA} , the

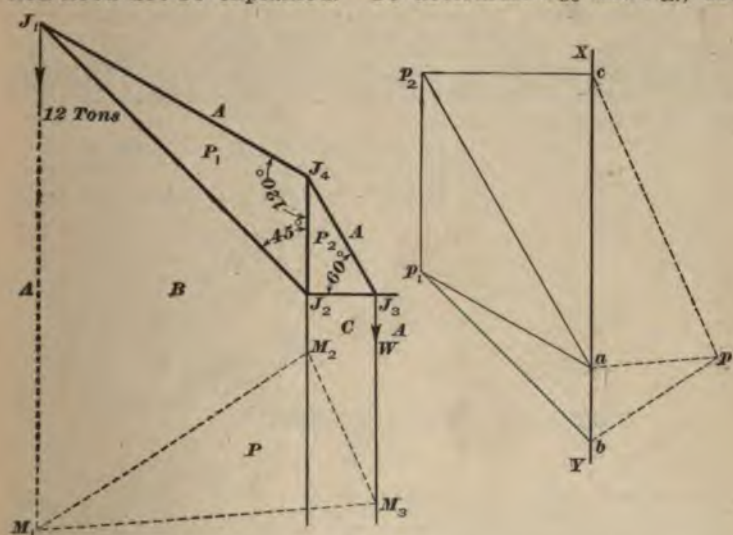


FIG. 18

method explained in Art. 17 is used. Having drawn an indefinite vertical line XY , the vector $ab = F_{AB} = 12$ tons is laid off anywhere on it, to a convenient scale. A pole p is chosen, and the rays pa and pb are drawn. From any point M_1 on AB , M_1M_2 , or PB , is drawn parallel to pb , meeting BC at M_2 ; M_1M_2 is drawn parallel to pa , meeting CA at M_3 . Through p , the line pc is drawn parallel to M_1M_3 , meeting XY at c . Then, $bc = F_{BC}$ and $ca = F_{CA} = W$.

Starting now with J_1 , the force polygon abp_1a is constructed in the usual manner by drawing bp_1 and ap_1 parallel, respectively, to BP_1 and AP_1 . The construction of the

Draw the vector $ab = F_{AB}$, assume the pole p , and draw the rays pa and pb ; also, ph , perpendicular to ab . From any point J on AB draw the strings PB and PA , and through the given point O draw $A'B'$ parallel to AB , meeting PA and PB at K and L , respectively. Draw JI perpendicular to $A'B'$ and AB , and, therefore, equal to ON . If the required moment is denoted by M , we may write,

$$M = F_{AB} \times ON = ab \times JI \quad (a)$$

The similar triangles JKL and pab give

$$\frac{ab}{ph} = \frac{KL}{JI}, \quad ab \times JI = ph \times KL$$

Comparing the second of these equations with (a), we get

$$M = ph \times KL \quad (b)$$

It is obvious that, so long as the pole p remains the same, the distance KL is independent of the position of the point J from which the strings are drawn. For, if any other point, as J_1 , is taken and the strings drawn as shown, intersecting $A'B'$ at K_1 and L_1 , the triangles $J_1K_1L_1$ and JKL are equal, and, therefore, $K_1L_1 = KL$. The distance KL varies according to the position of the pole; but, since M can have but one value, it follows from (b) that the product $ph \times KL$ is constant.

32. The Intercept.—The distance KL , Fig. 20, is called the *intercept* of the force F_{AB} , with respect to the point O and the pole p . The following general definition may, therefore, be given: The **intercept** of a force with respect to a given point O in the space diagram, and a given pole p in the force diagram, is the segment that the strings of the force intercept on a line drawn through the given point parallel to the line of action of the force.

33. The Normal Ray.—In the triangle abp , the vectors ap and pb represent the components of F_{AB} in directions parallel to AP and PB . Each of these components can be resolved into two resolutes: one along the line of action of the given force, and one perpendicular to that line. The resolutes of ap are ah and hp , and those of pb are ph and hb . The line hp represents, irrespectively of its direction, the

magnitude of either of the two resolutes perpendicular to AB , and will be called the **normal ray** of the force F_{AB} . This term is here introduced as more logical and consistent than the term *pole distance*, used by other writers on graphic statics.

34. Moment of a Force in Terms of the Intercept and the Normal Ray.—If the normal ray is denoted by F_n , and the intercept by i , equation (b) of Art. 31 may be written $M = F_n i$.

In words, *the moment of a force about a point is equal to the product of the normal ray and the intercept of the force, both referred to the same pole.*

The true magnitude of the normal ray is found by multiplying the length of the line ph by the scale of forces, and the true length of the intercept i is found by multiplying the length of KL by the scale of distances. Thus, if the forces are laid off to a scale of 100 pounds to the inch, and ph is found to measure 2.75 inches, then F_n will be $2.75 \times 100 = 275$ pounds. If the scale of distances is 20 feet to the inch, and KL is found to measure $\frac{9}{40}$ inch, then i will be $20 \times \frac{9}{40} = 4.5$ feet. These values in the formula give, $M = 275 \times 4.5 = 1,237.5$ foot-pounds.

35. Resultant Moment of Several Forces.—Since the resultant moment of several forces about any point is equal to the moment of the resultant of the forces about the same point, it can be found graphically by the method explained above, after the resultant of the forces has been determined. As an example, let it be required to find the resultant moment of the forces F_{AB} , F_{BC} , F_{CD} , and F_{DE} , Fig. 21, about the point O . The force polygon $abcdea$, constructed in the usual manner, gives the magnitude and direction of the resultant, represented by the vector ae . The funicular $J_1J_2J_3J_4J$ is next constructed. A parallel to ae drawn through J would be the line of action of the resultant; but it is not necessary to draw that line. The strings of the resultant are AP and PE , and they intercept the segment KL on a line drawn through O parallel to ae ; therefore,

KL is the intercept of the resultant. The normal ray of the resultant is ph , perpendicular to ac ; therefore, the required resultant moment is $ph \times KL$, care being taken before per-

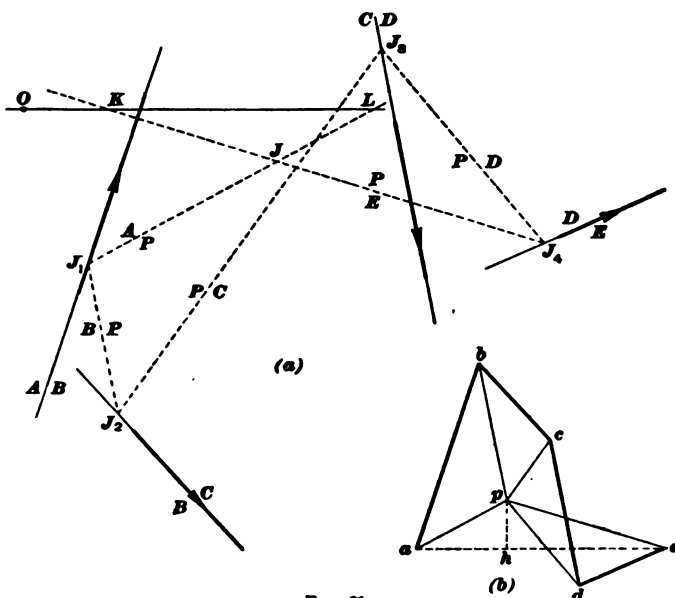


FIG. 21

forming the multiplication to make the proper reductions, according to the scales used.

PARALLEL FORCES

36. The Practical Problem.—The foregoing method finds its most common and useful application in the case of parallel forces, such as the weights acting on a structure. The problem, as it occurs in practice, may be stated in the following general terms:

Given a balanced system of parallel forces, required the resultant moment, about any point in the plane containing the forces, of all the forces on either side of the point.

Let F_{AB} , F_{BC} , etc., Fig. 22, be the given forces; $pabcd$, etc. their force diagram; and J , J , J , etc., the corresponding

string; in other words, PA and PE are the strings of the resultant of those forces, and, therefore, KL is the intercept of that resultant. The magnitude of the required moment is, therefore, $ph \times KL$. The magnitude and direction of the resultant of the forces under consideration are given by the vector ac , which shows that the resultant acts upwards (in the drawing). Its line of action, which passes through the intersection of PA and PE , lies, in this case, at the left of O . From these conditions, it follows that the resultant moment is right-handed.

The preceding conclusion as to the magnitude and direction of the moment can be directly arrived at as follows: Produce PB , PC , and PD to their intersections L_1 , L_2 , L_3 with OZ . Then, by definition, the intercepts of F_{AB} , F_{BC} , F_{CD} , and F_{DE} are, respectively, KL_1 , L_1L_2 , L_2L_3 , L_3L , and the moment of the resultant of the four forces is (paying due regard to signs),

$$+ph \times KL_1 - ph \times L_1L_2 - ph \times L_2L_3 + ph \times L_3L \\ = ph (KL_1 - L_1L_2 - L_2L_3 + L_3L) = +ph \times KL$$

as found before, the positive sign indicating that the resultant moment is right-handed.

37. Determination of the Intercept by the Funicular.—It follows from the foregoing that, *when several parallel forces are in equilibrium, the intercept, with respect to any point of their plane, of the resultant of all the forces on either side of the point, is the same as the segment intercepted by the funicular on a line drawn through the given point parallel to the common direction of the forces.*

This is a very useful rule, which finds its most important applications in problems relating to the strength of materials.

38. Selection of the Normal Ray.—Since the position of the pole in the force diagram is arbitrary, it is convenient so to select it that the normal ray will be a convenient number to multiply by. To accomplish this, it is sufficient to choose any point h in the force polygon (this applies to parallel forces only) and draw hp , normal to the vectors of the polygon, making it represent, to the scale of forces

used, a force of 100, 1,000, 10,000 units (pounds, tons, etc.) or some other number easy to be used as a multiplier. Thus, if the scale of forces is 500 pounds to the inch, p/h may be conveniently made equal to 2 or 4 or 6 inches, so that it will represent a force of 1,000, 2,000, or 3,000 pounds, respectively.

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STRESSES IN BRIDGE TRUSSES

(PART 1)

INTRODUCTION

DEFINITIONS AND GENERAL CONSIDERATIONS

BEAMS AND GIRDERS

1. Wooden and Steel Beams.—The principles governing the use of beams in supporting loads are fully discussed in *Strength of Materials*, Part 1. For short spans, beams are sometimes made of timber and are of solid rectangular cross-section, the same cross-section being used throughout the span. This is the most economical form when light beams can be used. The rectangular cross-section is not economical, however, when heavy beams are required; it must be made large enough to resist the maximum bending moment, and, as this occurs near the center of the span, some material is wasted by making the cross-section uniform, since no section between the center and the ends has to withstand so great a stress as the section at the center. For very heavy loads and long spans, it is more economical, when beams are used at all, to use steel beams. On account of the danger of fire and the frequent cost of renewal attending the employment of timber, steel beams are now almost exclusively used in permanent structures, even though their original cost may be many times that of wooden beams.

2. I Beams.—Where the bending moment is comparatively small and a low value of the section modulus is

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sufficient, rolled-steel beams of uniform cross-section are used, the material in the section being so distributed as to give a comparatively large value of the section modulus for a given amount of material. Since the top and bottom of the



FIG. 1

beam are farthest from the neutral axis, as much material as possible is concentrated in these parts. The beam whose cross-section is shown in Fig. 1 fulfills this condition, and, on account of its form, is called an **I beam**. The horizontal part at the top and bottom are called the **flanges**, and the vertical part is called the **web**. I beams are rolled in various sizes up to 24 inches in depth. For lengths not

exceeding 25 feet, the I beam is the most economical form of steel beam, but above this length too many rolled beams are required to resist the bending moment, so that a single deeper beam, made as explained in the next article, is more economical.

3. Plate Girders.—The name **plate girder** is given to a beam, usually of steel, that has the same general form as an I beam, but is composed of several pieces. The cross-section of such a beam is shown in Fig. 2. The vertical part consists of a plate called the **web**; while the top and bottom parts, which consist of plates and angles, are called the **flanges**. Plate girders are generally used for spans of 25 to 100 feet, and occasionally for greater lengths.



FIG. 2

The **span** of a plate girder is the horizontal distance from center to center of its supports; the **depth** is the vertical distance between the outer surfaces of the angles that form a part of the flanges. Experience shows that the most economical plate girder has a depth of $\frac{1}{8}$ from one-seventh to one-eighth of the span. The usual practice for railroad and highway bridges is to make the depth one-eighth to one-tenth of the span, although the depth is frequently made as small as one-twelfth. These latter proportions require heavier

sections than the former, and are, therefore, very wasteful of material. For spans over 100 feet in length, it is impossible to get sufficient depth for an economical section, and the girder is so heavy that it is difficult to handle. For these reasons, plate girders are unadaptable to spans over 100 feet in length, except under special conditions.

4. In the best modern practice, the top and bottom flanges of a plate girder are made parallel throughout the entire length of the girder, and are riveted to the web. The section of the flange is decreased from the center toward the end, and so no material is wasted, as the flanges at any point are just large enough to resist the maximum bending moment that can occur at that point. In the past, plate girders were built with curved flanges. The curve was usually made a parabola, and the flange cross-section was constant from end to end. There is no additional economy in the use of girders with curved or inclined flanges. Conditions, however, sometimes require that the ends be made shallower than the center, in which case one of the flanges is inclined near the end of the girder. The web of a plate girder is very thin and requires to be stiffened at intervals with angles riveted to the sides of the web to prevent it from buckling. This subject will be more fully treated elsewhere.

5. **Lattice Girder.**—The lattice girder is sometimes used for the same span length as the plate girder, and resembles the latter in that it has flanges, usually parallel, at the top and bottom, which decrease in size from the center toward the end of the girder. Instead of the solid web, however, there is an open-web system, consisting usually of angles running diagonally from top to bottom in both directions, and riveted to vertical plates that project from between the vertical legs of the flange angles. The lattice girder belongs to a special class of structures called *trusses*, which are designed to act as beams for any span length, but are most frequently and economically used for spans over 100 feet in length. Lattice girders, however, are somewhat used for highway bridges for spans much less than 100 feet.

THE TRUSS

6. Definition.—A truss may be defined as a framework composed of straight pieces, called **members**, so connected as to act, to a great extent, as a rigid structure. Trusses, like beams, are designed to support loads.

The intersection of two or more members, where they are connected or joined to each other, is called a **joint**.

While the truss as a whole resists the effect of the applied forces in much the same manner as the shear and the bending moment are resisted by a solid beam, each individual member of the truss is subjected only to direct tensile or compressive stresses in the direction of its length. In order that this may be the case, the applied forces are resolved into components acting at the joints of the truss.

The simplest form of truss is a triangle, and any truss is merely an assemblage of connected triangles. As the triangle is a rigid figure whose form cannot change so long as the length of each of its sides remains the same, it is the primary and essential element of the truss.

7. Truss Members.—The upper members of a truss, such as BC , CD , etc., Fig. 3, taken together form the

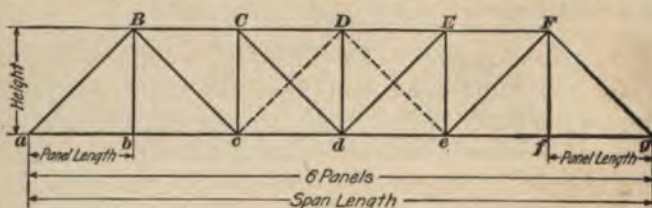


FIG. 3

upper chord of the truss; the lower members, such as ab , bc , etc., taken together form the **lower chord**. The other members of the truss, which lie between and connect the upper and lower chords, are called **web members**. The end web members aB and Fg are called **end posts**. The intermediate inclined web members, such as Bc and Cd , are called **diagonals**.

8. Panel, Span, and Height.—A panel is a subdivision of a truss between two consecutive joints in a chord. The joints are often called the **panel points**. The length of a panel is called the **panel length**. In Fig. 3, the points *b, c, d*, etc. are the joints or panel points, *ab* is the panel length, and the truss is a six-panel truss. Usually, the panel lengths of a truss are equal.

9. The **span** of a truss, for purposes of stress computation, is to be taken equal to the horizontal distance between the end panel points, as shown in Fig. 3.

10. The **height**, or **depth**, of a truss, for purposes of stress computation, is to be taken equal to the vertical distance between the joints of the chords, as shown in Fig. 3.

11. The three dimensions just given—panel length, span length, and depth of truss—bear a relation to each other that, to a certain extent, decides the type of truss to be used. Engineering experience has shown that for the greatest economy the depth of truss should be about one-sixth of the length, and that the diagonal web members should make an angle with the vertical of about 40° . The panel lengths most frequently used lie between 15 and 25 feet. These values need not be strictly adhered to; there may be a reasonable departure from them without seriously increasing the cost of the truss. From the span length, the depth and panel length are so chosen as to satisfy the economical conditions as far as possible.

12. Kinds of Trusses.—A **symmetrical truss** is a truss that can be cut at the center into two parts exactly alike. If it could be folded at the center on itself in such a manner that the two ends would come together, all corresponding members in the two halves of the truss would coincide. Nearly all trusses are symmetrical.

13. A **simple truss**, like a simple beam, is one that is supported only at the ends. A **continuous truss** and a **cantilever truss** are, likewise, similar to the corresponding forms of beams.

14. Parallel-chord trusses are trusses in which the upper and lower chords are parallel. In these trusses, the panel lengths are usually equal and all the diagonal web



FIG. 4

members have the same inclination (see Figs. 3 and 4). The parallel-chord truss is especially adapted to short spans.

15. In inclined-chord trusses, which are used for long spans, the depth at the center and the panel length are so chosen that the web members near the center make an angle with the vertical of less than 40° . The panels are made the same length throughout the bridge, but the depth



FIG. 5

of the truss is decreased at each panel point from the center toward the end, inclining one or both of the chords, thereby making the web members slope differently, those near the end making an angle with the vertical of more than 40° . This saves material near the end of the truss by shortening the heavy web members, and at the same time allows an economical depth to be used at the center of the truss.



FIG. 6

When the joints of the chord are in a straight inclined line, the chord is called an **inclined chord**, and the truss, an **inclined-chord truss**. When the joints of the chord are on a curve, the chord is called a **curved chord**, and the

truss, a **curved-chord truss**. The chord is not really curved, the members being straight between the joints. An inclined or a curved chord gives a graceful outline to a truss; on this account an inclined- or a curved-chord truss is



FIG. 7

preferable, from an esthetic standpoint, to a parallel-chord truss. Examples of inclined- and curved-chord trusses are shown in Figs. 5, 6, and 7.

16. Multiple-System Trusses.—For long spans, the chords are sometimes made parallel and the diagonal web members are continued across two or more panels. Trusses



FIG. 8

of this kind are called **multiple-intersection**, or **multiple-system**, trusses. When the diagonals extend over two panels, the truss is called a **double-intersection**, or **double-system**, truss; when the diagonals extend over three or four panels, the truss is called a **triple-** or **quadruple-system** truss, and also, to some extent, a **lattice**



FIG. 9

truss. The arrangement has also been used in trusses with curved and inclined chords.

The multiple-system truss allows an economical choice of both the center depth and the slope of the diagonals, without making it necessary to increase the length of panel.

Trusses of this type are objectionable, however, because the stresses cannot be found directly by the ordinary conditions

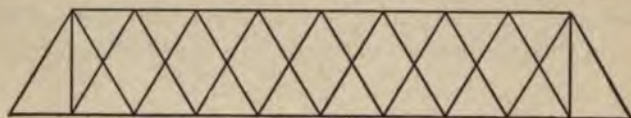


FIG. 10

of equilibrium. Examples of multiple-system trusses are shown in Figs. 8, 9, and 10.

17. Subdivided-Panel Trusses.—Another method of obtaining economy in the depth at the center and in the slope



FIG. 11

of the diagonals, without increasing the length of the panel, is to subdivide the main panels of the truss by adding secondary verticals, or secondary verticals and diagonals.

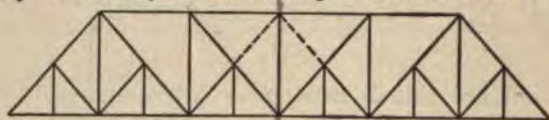


FIG. 12

This arrangement of members is made use of to a considerable extent in both parallel- and curved-chord trusses, and is very satisfactory. Any truss with subdivided panels is



FIG. 13

called a **subdivided-panel truss**. Examples of this type are shown in Figs. 11, 12, and 13.

18. The term **girder bridge**, or **truss bridge**, is the name usually applied to a bridge simply resting on supports at the same level, so that the reactions due to vertical

loads are vertical. This distinguishes the truss bridge from the arch and the suspension bridge, in which the reactions are not vertical.

19. Pin-connected trusses are those in which the several members that meet at a joint are connected to each other, and transmit their stresses, by means of an accurately turned pin, somewhat resembling a large bolt, which is made to fit very closely into holes drilled through the ends of the members. This affords a simple and convenient method of connecting the members. As the connection acts, to some extent, like a large hinge, it allows each member to adjust itself in the line of its stress without developing large bending stresses. In this type of truss, each member is practically finished at the shop; and, in erecting the bridge at its site, the members are assembled and connected, and the structure completed, in the shortest possible time and with the minimum amount of field labor. Owing largely to this fact, this type of truss is the standard in America for long spans. The pin-connected truss is well adapted for both highway and railroad bridges over 150 feet in length.

20. Riveted trusses are those in which the several members that meet at a joint are riveted to each other, and transmit their stresses by means of connecting plates called **gussets**, to which all the members at the joint are riveted. The tendency of the best modern practice is toward such details as will give great rigidity, and for this reason the riveted truss, which is very rigid, is coming into general use for the shorter spans to which trusses are adapted. It is used for highway and for railroad bridges for spans up to about 150 feet in length.

21. Line of Action of Stress in a Truss Member. The stress in each member of a truss is considered to act in a direct line between the centers of the connections. The member itself should, therefore, be straight, and the connections at its extremities should be as nearly as possible on a line passing through the center of gravity of its cross-section. The pins in a pin-connected truss are located on lines passing

nearly through the centers of gravity of the members, and the stresses are therefore all direct stresses. There is some eccentricity in the connections of a riveted truss, due to the fact that the members are connected to large gusset plates; and, although the center lines of all the members at any joint intersect in the same point, the centers of the connections are not coincident; this causes eccentric stress.

22. Stress Sheet.—For all purposes relating to the investigation of the stresses in the members of a truss, each member is represented by a straight line indicating the line of action of its stress. A **stress sheet** of a bridge is a skeleton drawing in which the members of the bridge are shown by straight lines, which occupy positions on the drawing corresponding to the positions of the members in the bridge. On the stress sheet are shown the arrangement of floor and lateral systems, with the spacing of stringers and trusses; the span length, number of panels, panel length, and depth of truss; the lengths of diagonals; the assumed dead, live, and wind loads, and the panel loads and reactions resulting therefrom; and the maximum and minimum stresses in each member due to the assumed loads. The amount of material required in each member, and the different parts that form the cross-section of the member, are sometimes shown on the stress sheet.

TRUSS AND PLATE-GIRDER BRIDGES

MAIN PARTS

23. The main parts of a truss bridge or of a plate-girder bridge are: (1) two longitudinal vertical trusses or two-plate girders, such as have been described in the foregoing articles; (2) a *floor system*; (3) a *lateral system*.

24. The Floor System.—The traffic on a bridge is carried by a floor, which transfers the load to the longitudinal trusses or girders on the sides. In a highway bridge, the floor usually consists of planking resting on longitudinal

joists, or **stringers**, and of cross-girders called **floorbeams**, which support the stringers and transfer the load to the trusses at the panel points. In a railroad bridge, the floor usually consists of wooden rail ties, longitudinal stringers, and floorbeams. In some truss bridges, the floor joists, or the ties, rest directly on the chord members, thus producing bending stresses in these members in addition to the direct stresses. These bending stresses must be separately computed and added to the direct stresses. This subject, however, will be left for subsequent treatment; it will here be assumed that the floor system transfers the load to the trusses at the joints, and that, therefore, the load acts at these points only. The complete analysis of a bridge includes the analysis of the stresses in the various parts of the floor system, as well as in the members of the trusses.

25. The Lateral System.—The lateral forces acting on a bridge are resisted by lateral trusses lying in the planes of the chords and connected to the latter; these trusses transmit all lateral forces to the supports. In addition to this, there are transverse braces, or frames, at each panel point, which are made as deep as the conditions will allow. The transverse brace, or frame, at the end is placed in the plane of the end posts and is usually called the **portal brace**, or simply the **portal**.

CLASSIFICATION OF BRIDGES

26. There are several ways in which bridges may be classified. One of the most common is to classify them according to the position of the floor or roadway with respect to the chords. In this classification, bridges are divided into three general types, namely: *through bridges*, *deck bridges*, and *half-through bridges*.

27. Through bridges are those that support their floors or loads at or near the level of the bottom chord, and have room for a system of lateral bracing between the top chords without interfering with the traffic. The loads pass between the trusses, or *through* the bridge. Bridges of this type

require very little space below the floor, and, therefore, the locations to which they are adapted are very common. Where trusses are of such a height that vertical transverse frames may be put in between the web members above the overhead clearance line, they are sometimes spoken of as *high-truss bridges*.

28. Deck bridges are those that support their floors or loads at or near the level of the upper chord. In bridges of this class, all portions of the structure are entirely below the floor. For long spans, the locations to which they are adapted are not very common. Deck bridges are more economical than any other type of bridge, and are used wherever conditions permit.

29. The chord that supports the floor system is called the **loaded chord**; the other, the **unloaded chord**. In a through bridge, the loaded chord is the lower chord; in a deck bridge, the loaded chord is the upper chord.

30. Half-through bridges are those that support their floors or loads at some elevation intermediate between the top and the bottom chord, or at the bottom chord, and the trusses of which are not deep enough to allow a system of overhead bracing. When plate girders are used, such a bridge is called a half-through plate-girder bridge. When trusses are used, such a bridge is called a **low-truss**, or **pony-truss, bridge**.

31. Bridges are also spoken of as *plate-girder bridges*, *riveted-truss bridges*, and *pin-connected truss bridges*, according to the type of beam or truss that supports the loads. Bridges may be further classified according to the style of truss, giving two general classes; namely, **parallel-chord bridges** and **inclined-chord bridges**. Each of these classes may be subdivided into single-system, multiple-system, and subdivided-panel bridges, as explained in connection with trusses.

LOADS AND REACTIONS

CLASSIFICATION OF LOADS

32. External Forces Acting on a Bridge.—The external forces acting on a bridge consist of: (1) the weight of the structure itself; (2) the weight of whatever the structure is designed to support; (3) the pressure of the wind; and (4) the reactions of the abutments. The first two of these classes of forces are called **loads**. In addition to the forces just mentioned, it is sometimes necessary to consider other applied forces, such as the centrifugal force of a train moving on a curved track, and the horizontal force caused by moving trains.

33. Kinds of Loads.—Loads are divided into two general classes, namely: *dead loads* and *live loads*.

The **dead load** consists of the weight of the structure itself, including the track or floor, the floor system, and the girders or trusses. It is the force of gravity acting on every part of the structure, and is, therefore, actually applied at all points. The weight of the floor system is transferred to the joints of the loaded chord by the floor-beams. The weight of the truss is transferred to the joints of the loaded and unloaded chords by the members themselves. Methods of estimating in advance the approximate weight of floor systems and trusses will be given elsewhere. The dead load is usually assumed to be a uniform amount per linear foot of structure. For short spans, up to about 125 feet, the direct stresses due to dead load are found by assuming all the load to be applied at the joints of the loaded chord. For longer spans, a part of the load, usually one-third, is assumed to act at the joints of the unloaded chord. This assumption leads to results that are very close to the actual stresses.

34. The **live load** is the load due to the traffic. It is therefore a **moving load**, and is often so called. For highway bridges, the live load is assumed to be a specified amount per square foot, uniformly distributed over the roadway. For bridges that are subject to the passage of heavy loads concentrated on wheels, the live load is assumed to be a certain arrangement of concentrated loads, or wheel loads, which may be taken by themselves or in connection with the uniform load. For railroad bridges, the live load usually consists of a system of concentrated wheel loads representing a type of locomotive, or of certain uniform loads that will give an approximately equivalent effect. The calculation of stresses due to uniform and to concentrated loads require separate consideration. This subject will be fully treated elsewhere. The amount of live load that a bridge is designed to carry is sometimes called the **capacity** of the bridge.

35. Wind Pressure.—The wind pressure, sometimes called the **wind load**, is the force exerted by the wind on all surfaces exposed to it. These surfaces consist of the sides of the members and the exposed surfaces of the moving loads that cross the structure. In highway bridges, the surface of the loads is usually very small compared to the exposed surface of the bridge. In railroad bridges, it is necessary to consider the pressure of the wind against the side of a train, in addition to the pressure against the exposed surface of the structure. It is customary to assume the wind pressure as a uniformly distributed force, and express it in pounds per square foot of exposed surface, or in pounds per linear foot of the loaded or unloaded chord.

36. Centrifugal Force.—The centrifugal force considered in bridges is the pressure exerted by a train of cars moving on a curved track; it acts radially outwards, and is transferred by the floor system to the joints of the loaded chord. It is usually expressed as a percentage of the live load and depends on the degree of curvature of the track, and on the weight and speed of the train.

37. The longitudinal thrust is the force exerted on a structure by a train of cars crossing the structure while the brakes are set. This force is a maximum when the brakes are set hard enough to prevent the wheels from turning, in which case they slide on the rails.

The **tractive force** is the force exerted on a structure by the friction of the driving wheels of a locomotive drawing a train of cars; it is usually less than the longitudinal thrust, and may be neglected if the latter is taken into consideration.

PANEL LOADS

38. Definition.—The amount of load that is transferred to a joint of the loaded chord is called a **panel load**, or **panel concentration**.

39. Dead Panel Loads.—As the weight of the floor system is a large part of the dead load, the dead panel loads of the loaded chord are larger than those of the unloaded chord. If all the dead load were considered as applied along the loaded chord, each panel load would equal the uniform load per linear foot multiplied by the length of one panel and divided by 2 (there being two trusses to the bridge). It is convenient to compute first this panel load, and then, if considered necessary, assume an approximate distribution of it between the loaded and the unloaded chord, as explained in Art. 42.

40. Live Panel Loads.—All the live load is applied along the loaded chord. In highway bridges, the load per linear foot is equal to the specified load per square foot on the floor, multiplied by the clear width of the roadway, plus a like product for the sidewalks, if any. The panel load is then equal to one-half this total uniform load multiplied by a panel length. Some highway bridges have only one sidewalk, which is sometimes supported outside of the truss. In this case, the load on and stresses in each truss must be computed separately. In railway bridges, if the load is a uniform load per linear foot, the panel load is found as just

stated; if it consists of a series of wheel loads, the panel loads will vary in amount. The treatment of this class of loads is considered in a subsequent Section.

41. Wind Panel Load.—The wind pressure per linear foot, assumed or computed for either chord, multiplied by the panel length gives the wind panel load for that chord.

42. Illustrative Example.—As an example, let a through highway bridge, such as is shown in Fig. 14, contain seven equal panels, each 20 feet long, and have a clear width of roadway of 16 feet. Suppose the dead load to be 600 pounds per linear foot, and the live load to be 100 pounds

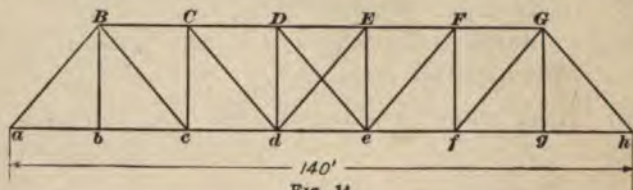


FIG. 14

per square foot of roadway. The dead panel load will then be

$$\frac{600 \times 20}{2} = 6,000 \text{ pounds}$$

and the live panel load,

$$\frac{100 \times 16 \times 20}{2} = 16,000 \text{ pounds}$$

Assuming all the dead load to be applied at the joints of the loaded chord, each of the joints *b, c, d, e, f,* and *g* will be loaded with 6,000 pounds dead load; while, if the bridge sustains a live load throughout its length, each of the joints will also have a load of 16,000 pounds. At the points *a* and *h*, there is a half-panel load consisting of 3,000 pounds dead load and 8,000 pounds live load. These half-panel loads are carried directly by the supports and do not affect the trusses, and hence can be omitted in the calculation of stresses. A seven-panel truss would thus be considered as loaded with six intermediate panel loads; a six-panel truss would likewise have five intermediate panel loads; etc. If it is desired to distribute the dead load between the lower and

the upper chord, one-third of the load, or 2,000 pounds, may be assumed as acting at each of the upper joints, and 4,000 pounds at each of the lower joints, except at a and h , each of which would still carry the half-panel load of 3,000 pounds.

EXAMPLES FOR PRACTICE

1. A bridge 99 feet long is designed to sustain a live load of 100 pounds per square foot on a roadway 16 feet wide, clear width. The trusses are divided into 6 panels. What is: (a) the live load per linear foot? (b) the panel live load?

Ans. $\begin{cases} (a) & 1,600 \text{ lb.} \\ (b) & 13,200 \text{ lb.} \end{cases}$

2. An eight-panel bridge of 120 feet span carries a roadway 18 feet wide, clear width. The live load assumed for the trusses is 96 pounds per square foot of roadway. What is: (a) the live load per linear foot? (b) the panel live load?

Ans. $\begin{cases} (a) & 71,728 \text{ lb.} \\ (b) & 12,960 \text{ lb.} \end{cases}$

3. If for the bridge of example 2 the wind load per linear foot is assumed as 300 pounds for the lower chord and 150 pounds for the upper chord, what is the panel wind load: (a) for the lower chord? (b) for the upper chord?

Ans. $\begin{cases} (a) & 4,500 \text{ lb.} \\ (b) & 2,250 \text{ lb.} \end{cases}$

4. Suppose, for the bridge described in example 2, that the dead load is assumed to be 760 pounds per linear foot. What will the panel dead load be?

Ans. 5,700 lb.

REACTIONS

43. In all bridge trusses, one end is free to move horizontally in a longitudinal direction, so that the reactions due to the dead and live loads will be vertical. The effect of wind pressure and other horizontal forces will receive separate consideration.

44. **Dead-Load Reactions.**—By the principles of statics, the reactions exerted by the supports must hold in equilibrium the applied loads. Let Fig. 15 represent any truss acted on by the panel loads W as shown. Let R_1 and R_2 be the reactions. The loads and reactions together comprise all the external forces acting on the structure, and any of the conditions of equilibrium may be applied to these forces. The object being to determine the values of the reactions R_1 and R_2 , it will be convenient to take moments

of the forces about the point f . The following equation therefore obtains:

$$\Sigma M_f = R_1 \times 100 - W \times 80 - W \times 60 - W \times 40 - W \times 20 = 0$$

whence
$$R_1 = \frac{W \times 200}{100} = 2W$$

In like manner, by taking moments about a , the reaction R_2

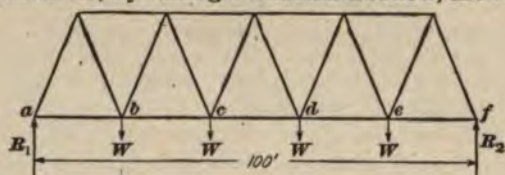


FIG. 15

is found to be $2W$. Or, by putting the sum of the vertical forces equal to zero,

$$R_1 - 4W + R_2 = 0;$$

whence, as before,

$$R_2 = 4W - R_1 = 4W - 2W = 2W$$

In this case, where the load is symmetrical, it is evidently unnecessary to write out these equations, since the reactions must be equal, and each must be equal to one-half the total load, or to $2W$.

45. Live-Load Reactions.—If the truss is only partly loaded, as in Fig. 16, the reactions are not equal, and it is necessary to calculate them by applying the principles of

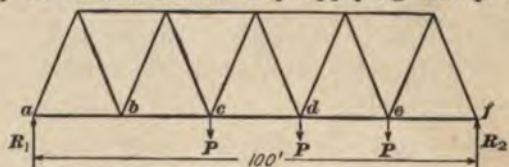


FIG. 16

statics as just stated. In the truss shown, with f as a center of moments, we have

$$\Sigma M_f = R_1 \times 100 - P \times 60 - P \times 40 - P \times 20 = 0; \quad (1)$$

whence
$$R_1 = \frac{P \times 120}{100} = 1.2P$$

Also,
$$R_1 + R_2 - 3P = 0;$$

whence
$$R_2 = 3P - R_1 = 3P - 1.2P = 1.8P$$

This method of calculation is perfectly general and applies as well when the panel loads or panel lengths are unequal.

46. Where the panel lengths are equal, as is usually the case, the reactions for a partial load, as in Fig. 16, can be found more readily by writing equation (1), Art. 45, in a little different form. Taking the panel length as the unit of length, and changing the order of arrangement, we have

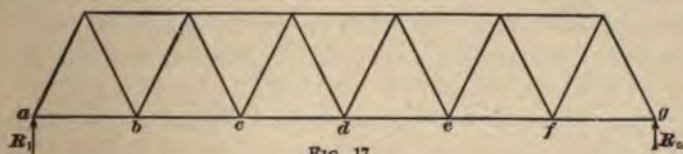
$$R_1 \times 5 - P \times 1 - P \times 2 - P \times 3 = 0;$$

$$\text{whence} \quad R_1 \times 5 = 1P + 2P + 3P$$

$$\text{and} \quad R_1 = \frac{1}{5}P + \frac{2}{5}P + \frac{3}{5}P$$

Thus, it is seen that the left reaction is equal to one-fifth the load at *e*, plus two-fifths the load at *d*, plus three-fifths the load at *c*. This statement can be written at once by inspection; for, as regards the load at *e*, it is known, from the principle of moments, that one-fifth will be carried by the left support, and four-fifths by the right; likewise, two-fifths of the load at *d* will be carried at *a* and three-fifths at *f*, etc. This method of getting reactions for partial loads, *when the panel lengths are equal*, is very convenient and will be frequently employed hereafter.

EXAMPLE.—(a) Assuming the truss shown in Fig. 17 to be loaded at each of the lower chord joints with a load of 5,000 pounds, to cal-



culate the reactions. (b) Assuming the joints *d*, *e*, and *f* only, to be loaded with a load of 5,000 pounds each, to calculate the reactions.

SOLUTION.—(a) In this case, the load is symmetrical; therefore,

$$R_1 = R_2 = \frac{5 \times 5,000}{2} = 12,500 \text{ lb. Ans.}$$

(b) Taking moments about *g*,

$$R_1 \times 6 - 5,000 \times 1 - 5,000 \times 2 - 5,000 \times 3 = 0;$$

whence

$$R_1 = \frac{5,000 \times 1 + 5,000 \times 2 + 5,000 \times 3}{6} = 5,000 \text{ lb. Ans.}$$

Taking moments about a

$$- R_s \times 6 + 5,000 \times 3 + 5,000 \times 4 + 5,000 \times 5 = 0;$$

whence

$$R_s = \frac{5,000 \times 3 + 5,000 \times 4 + 5,000 \times 5}{6} = 10,000 \text{ lb.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. If the truss shown in Fig. 17 carries a panel load of 6,000 pounds at each of the lower chord joints, what are the two reactions?

$$\text{Ans. } \begin{cases} R_1 = 15,000 \text{ lb.} \\ R_s = 15,000 \text{ lb.} \end{cases}$$

2. If the truss shown in Fig. 17 carries a panel load of 6,000 pounds at each of the joints b , c , and d , what are the two reactions?

$$\text{Ans. } \begin{cases} R_1 = 12,000 \text{ lb.} \\ R_s = 6,000 \text{ lb.} \end{cases}$$

SHEARS AND MOMENTS

MAXIMUM MOMENTS AND SHEARS IN BEAMS

BENDING MOMENTS

47. It is desirable here to review some of the principles explained in *Strength of Materials*, Part 1, in connection with beams, and to add sufficient further discussion to make complete the analysis of maximum moments and shears in simple beams and trusses for fixed and for moving uniform loads. In designing a beam, it is necessary to know the maximum bending moments and the maximum shears at various sections. From the former, the flange or fiber stresses are found; and from the latter, the shearing or web stresses. It will be sufficient here to deal with this subject in a general manner, stating the fundamental principles. The application of these principles to the calculation of the actual flange and web stresses will be taken up in connection with design, where it can be more conveniently treated.

48. **Bending Moments Due to a Uniform Dead Load.** As explained in *Strength of Materials*, Part 1, the bending moment at any section of a beam is the sum of the

moments of all the external forces to the left or right of the section about the neutral axis of the section. In Fig. 18 (a), AB is a beam of length l , loaded with a uniform load of

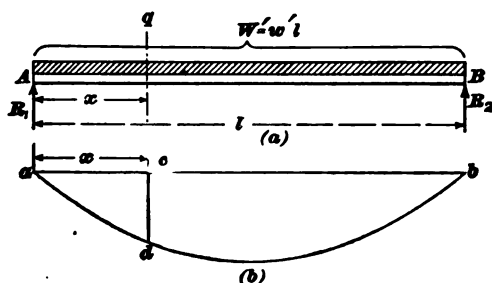


FIG. 18

w' units of weight per unit of length. The total load is $w' l$, and each reaction is equal to $\frac{w' l}{2}$. The bending moment M' at any section q , distant x from the left end, is given by the formula

$$M' = R_1 x - w' x \times \frac{x}{2};$$

or, substituting the value of R_1 , and factoring,

$$M' = \frac{w'}{2}(l - x) x \quad (1)$$

This moment is a maximum at the center, where its value is given by the formula

$$\text{Max. } M' = \frac{w' l^2}{8} \quad (2)$$

The bending moments at all points of the beam due to this uniform load are represented graphically in Fig. 18 (b) by the moment curve adb , which is a parabola and shows how the moment varies along the beam.

49. Bending Moment Due to a Uniform Live Load.

The maximum bending moment at any section of a simple beam, caused by a uniform live load, will occur when the load extends entirely across the span. The beam being fully loaded for maximum moments, these moments will be given by formula 1 of the preceding article, except that w'

should be replaced by the live load w'' per unit of length. If, therefore, the maximum live-load moment at the point q is denoted by M'' , then

$$M'' = \frac{w''}{2} (l - x) x \quad (1)$$

From the preceding article,

$$M' = \frac{w'}{2} (l - x) x$$

Dividing the first by the second of these two equations, the result is

$$\frac{M''}{M'} = \frac{w''}{w'};$$

whence

$$M'' = M' \times \frac{w''}{w'} \quad (2)$$

When, therefore, the dead-load moment at any section of the beam has been determined, the maximum live-load moment at the same section is obtained by multiplying the dead-load moment by the ratio $\frac{w''}{w'}$ of the live load per unit of length to the dead load per unit of length.

If the total or resultant bending moment at q is denoted by M , then

$$M = M' + M'';$$

or, substituting the values of M' and M'' ,

$$M = \frac{w' + w''}{2} (l - x) x \quad (3)$$

EXAMPLE.—A beam 40 feet long supports a dead load of 500 pounds per foot and a live load of 1,800 pounds per foot. What are the dead-load and maximum live-load bending moments at points 10 feet apart along the beam?

SOLUTION.—The moments will be found by means of formula 1, Art. 48, and formula 2, Art. 49. Here, $l = 40$, $w' = 500$ lb., $w'' = 1,800$ lb., and

$$\frac{w''}{w'} = \frac{1,800}{500} = 3.6$$

The values of x for the several points are 0, 10, 20, 30, and 40. The moments, in foot-pounds, are as follows:

$$\text{For } x = 0, M' = \frac{500}{2} (40 - 0) 0 = 0, M'' = 0.$$

$$\text{For } x = 10, M' = 250 (40 - 10) 10 = 75,000, M'' = 75,000 \times 3.6 = 270,000.$$

For $x = 20$, $M' = 250(40 - 20)20 = 100,000$, $M'' = 100,000 \times 3.6 = 360,000$.

For $x = 30$, $M' = 250(40 - 30)30 = 75,000$, $M'' = 75,000 \times 3.6 = 270,000$.

For $x = 40$, $M' = 250(40 - 40)40 = 0$, $M'' = 0$.

EXAMPLES FOR PRACTICE

1. A beam 100 feet long supports a dead load of 600 pounds per foot. What is the dead-load moment: (a) at the center? (b) at a point 75 feet from the left end?

Ans. $\begin{cases} (a) & 750,000 \text{ ft.-lb.} \\ (b) & 562,500 \text{ ft.-lb.} \end{cases}$

2. A beam 40 feet long supports a live load of 750 pounds per foot. What is the live-load moment at a point: (a) 10 feet from the left support? (b) 20 feet? (c) 30 feet?

Ans. $\begin{cases} (a) & 112,500 \text{ ft.-lb.} \\ (b) & 150,000 \text{ ft.-lb.} \\ (c) & 112,500 \text{ ft.-lb.} \end{cases}$

3. A beam 80 feet long supports a dead load of 450 pounds per foot and a live load of 1,200 pounds per foot. What is: (a) the dead-load bending moment, and (b) the live-load bending moment at a point 30 feet from the left support?

Ans. $\begin{cases} (a) & 337,500 \text{ ft.-lb.} \\ (b) & 900,000 \text{ ft.-lb.} \end{cases}$

4. In example 3, what is the total bending moment at the center of the beam?

Ans. 1,320,000 ft.-lb.

SHEARS

50. Shear and Its Sign.—The shear at any section of a beam is defined as the sum of all the vertical forces acting on the beam to the left or right of the section. The correct

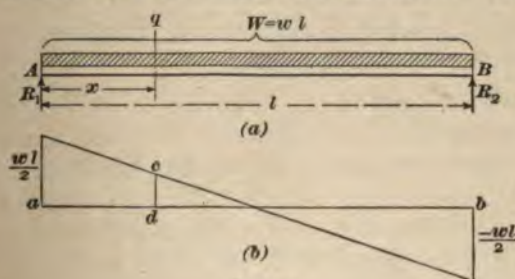


FIG. 19

sign will be given by considering the forces on the left and taking upward forces as positive and downward forces as

negative. If the load is a uniform load w per unit of length, Fig. 19 (a), the shear at any point whose distance from the left end is x is given by the formula

$$V = R_1 - wx = \frac{wl}{2} - wx$$

At the left end of the beam, where $x = 0$, the shear is equal to R_1 , or $\frac{wl}{2}$; at the center, where $x = \frac{l}{2}$, the shear is 0; and at the right end, where $x = l$, the shear is $-\frac{wl}{2}$, or $-R_1$. The shear diagram for dead load is shown in Fig. 19 (b).

It is to be noted that the shears on the right of the center are negative, and those on the left are positive, but that for two points equidistant from the center the shears are of the same numerical value.

51. The meaning of positive and negative shears will be made clearer by a study of Fig. 20, which shows how the forces acting tend to shear the beam at any given section. *Positive shear* at any section q occurs when the external forces tend to move the left-hand portion upwards and the right-hand portion downwards; *negative shear* occurs when the external forces tend to move the left-hand portion downwards and the right-hand portion upwards.

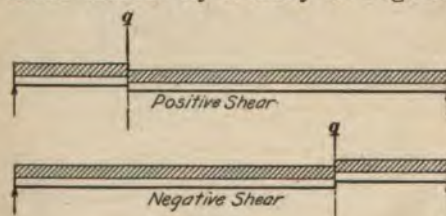


FIG. 20

52. **Live-Load Shears.**—In order to arrive at a general rule for determining the maximum live-load shear at any section of a beam, it will be convenient to consider first the effect of a single load W , Fig. 21, placed anywhere on the beam. Let q be the section at which the shear is required. Considering the weight W first at any point on the *right* of the section, it is seen that the only external force acting on the left of the section will be the reaction. The shear at the section will then be equal to this reaction, and will be *positive*.

If the load W is placed anywhere to the *left* of the section, the shear will be equal to the left reaction *minus* the load W ; and, as the reaction is always less than the load, the resultant of the reaction and the load will act *downwards*, and the shear will be *negative*.

It is thus seen that a load placed anywhere on the right of a section will cause positive shear in that section, and if placed

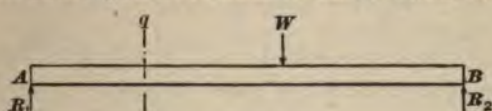


FIG. 21

anywhere on the left of the section, it will cause negative shear. From this it follows that, if a number of loads is to be placed on a beam so as to cause the maximum positive shear at a given section, as many loads as possible should be placed on the right of the section, and none on the left. For maximum negative shear, the reverse would be true. If the given load is a uniform live load, the maximum positive shear at any section obtains when the beam is loaded on the right of the section only, and the maximum negative shear, when the beam is loaded on the left.

53. Applying the rule just given to any beam AB , Fig. 22 (a), the maximum positive live-load shear at any

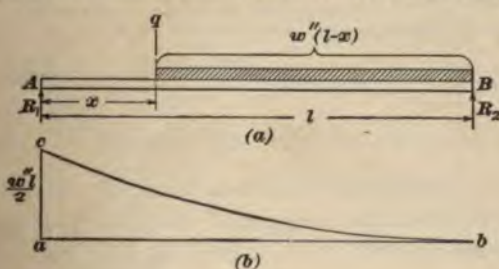


FIG. 22

section q , distant x from the left end, due to a uniform load of w'' per unit of length, obtains when the load covers all the part of the beam to the right of q . The shear, which will be denoted by V'' , will be equal to the left reaction. Taking

moments about B , and noting that the total load on the beam is equal to $w''(l-x)$, and that its lever arm about B is equal to $\frac{l-x}{2}$, the following equation obtains:

$$R_1 \times l - w''(l-x) \times \frac{l-x}{2} = 0;$$

whence
$$R_1 = \frac{w''(l-x) \left(\frac{l-x}{2} \right)}{l} = \frac{w''}{2l} (l-x)^2$$

As the maximum live-load shear is equal to R_1 , then

$$V'' = \frac{w''}{2l} (l-x)^2$$

If the load covers the whole beam, $x = 0$, and

$$V'' = \frac{w'' l^2}{2l} = \frac{w'' l}{2}$$

If the load covers half the beam, $x = \frac{l}{2}$, and, therefore,

$$V'' = \frac{w''}{2l} \left(l - \frac{l}{2} \right)^2 = \frac{w''}{2l} \left(\frac{l}{2} \right)^2 = \frac{w''}{2l} \times \frac{l^2}{4} = \frac{w'' l}{8}$$

In Fig. 22 (b), the ordinates to the curve cb represent the

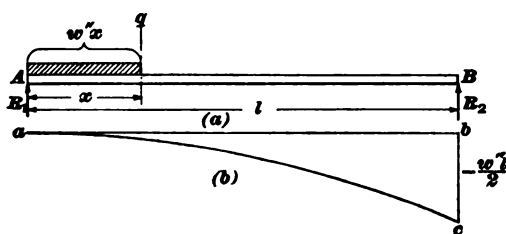


FIG. 23

maximum positive shears in the beam at all points, obtained by giving to x values between l and 0 . This curve is a parabola with its vertex at b , and may be constructed by the methods explained in *Rudiments of Analytic Geometry*.

54. The maximum live-load negative shear at any point in the beam will occur when the beam is loaded on the left at that point, as shown in Fig. 23 (a). The maximum shear at section g will equal the left reaction minus the load $w''x$.

The left reaction R_1 is found by taking moments, as before, about the point B :

$$R_1 = \frac{w''x \left(l - \frac{x}{2} \right)}{l} = w''x - \frac{w''x^2}{2l}$$

and the maximum negative shear is

$$V'' = R_1 - w''x = -\frac{w''x^2}{2l}$$

This value is equal to the right reaction, and might have been found by taking moments about A , without previously computing R_1 . Fig. 23 (*b*) shows the shear diagram for negative shears.

By examining the values of the negative shears, beginning at the right end and passing toward the left, it will be observed that the maximum positive shear at any point is numerically equal to the maximum negative shear at a point in the opposite end of the beam equidistant from the center.

55. Illustrative Example.—Let it be required to determine the dead-load shears at sections 5 feet apart in a beam 60 feet long, Fig. 24. Let the dead load be 400 pounds

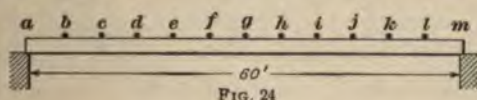


FIG. 24

($= w'$) per linear foot, and the live load 1,200 pounds ($= w''$) per linear foot.

1. *Dead-Load Shears.*—The reactions are each equal to

$$\frac{400 \times 60}{2} = 12,000 \text{ pounds}$$

The shears will be found by the formula of Art. 50, substituting w' , or 400, for w .

At a , $x = 0$, and $V' = R_1 = +12,000$ pounds.

At b , $x = 5$, and $V' = 12,000 - 400 \times 5 = +10,000$ pounds.

At c , $x = 10$, and $V' = 12,000 - 400 \times 10 = +8,000$ pounds.

In like manner, the shear for each of the other given sections is found. The results are given in the second column of the accompanying table.

TABLE OF SHEARS

1	2	3	4	5	6
Section	Dead-Load Shear	Maximum Live-Load Shear		Combined Shear	
		Positive	Negative	Dead-Load and Maximum Positive Live-Load Shear	Dead-Load and Maximum Negative Live-Load Shear
<i>a</i>	+12,000	+36,000	0	+48,000	+12,000
<i>b</i>	+10,000	+30,250	- 250	+40,250	+ 9,750
<i>c</i>	+ 8,000	+25,000	- 1,000	+33,000	+ 7,000
<i>d</i>	+ 6,000	+20,250	- 2,250	+26,250	+ 3,750
<i>e</i>	+ 4,000	+16,000	- 4,000	+20,000	0
<i>f</i>	+ 2,000	+12,250	- 6,250	+14,250	- 4,250
<i>g</i> (center)	0	+ 9,000	- 9,000	+ 9,000	- 9,000
<i>h</i>	- 2,000	+ 6,250	-12,250	+ 4,250	-14,250
<i>i</i>	- 4,000	+ 4,000	-16,000	0	-20,000
<i>j</i>	- 6,000	+ 2,250	-20,250	- 3,750	-26,250
<i>k</i>	- 8,000	+ 1,000	-25,000	- 7,000	-33,000
<i>l</i>	-10,000	+ 250	-30,250	- 9,750	-40,250
<i>m</i>	-12,000	0	-36,000	-12,000	-48,000

It is to be noted that the shears beyond *g*, the center section, may be written out at once from the values to the left of this section.

2. *Live-Load Shears*.—The maximum positive live-load shears are obtained from the formula of Art. 53. At *a*, where $x = 0$,

$$V'' = R_1 = \frac{1,200 \times 60}{2} = + 36,000 \text{ pounds}$$

At *b*, $x = 5$, and

$$V'' = \frac{1,200}{2 \times 60} (60 - 5)^2 = + 30,250 \text{ pounds}$$

At *c*, $x = 10$, and

$$V'' = \frac{1,200}{2 \times 60} (60 - 10)^2 = + 25,000 \text{ pounds}$$

and so on. These shears are given in the third column of the above table. The maximum negative live-load shears are found in like manner, by the use of the formula in Art. 54. The fifth and the sixth column of the table contain the combined shears, found as explained below.

3. *Combined Shears.*—Since the dead load is always present, the maximum total shear at any section of the beam will be found by combining the dead-load shear with one or the other of the maximum live-load shears. Combining with the maximum positive shears, the fifth column of the preceding table is obtained; while the sixth column is obtained by combining the dead-load shears with the maximum negative live-load shears. By comparing these two columns, it will be noticed that, for sections to the left of the center, the maximum shears are the positive shears of column 5; while to the right of the center they are the negative shears of column 6. For two sections equidistant from the center, the maximum shears are numerically equal.

It is to be further noted that in column 5 there is a small positive shear at the sections *h*, zero shear at *i*, and negative shears beyond. These negative shears, which are smaller than the dead-load shears, are equal to the difference between the dead-load negative and the live-load positive shears. They are in fact the least negative shears that can occur in the beam. In the same way, column 6 gives, in the upper part, the least positive shears down as far as *e*, then the maximum negative shears for the remainder of the beam. Taking the two columns together, the greatest *range* of shear is obtained that can possibly occur in the beam at the several sections for any positions of the live load. Thus, at section *a*, the shear may be as high as + 48,000 and as low as + 12,000; at *b*, the limits are + 40,250 and + 9,750; at *c*, + 20,000 and 0; at *f*, + 14,250 and - 4,250; and at *g*, + 9,000 and - 9,000. On the right end, the limits are the same numerically at corresponding sections, but are of opposite sign. From this discussion, it is plain that only positive shears can exist from *a* to *e*; both kinds are possible from *e* to *i*; and only negative shears beyond *i*. This table should

be carefully studied, as the relations here illustrated are of great importance in the analysis of trusses.

4. *Summary of Results.*—Summarizing the results of the foregoing analysis, the following statement may be made regarding a beam subjected to both a dead and a uniform live load: (1) The *maximum shear* at any section to the left of the center will be *positive*, and may be found by adding to the dead-load shear the maximum positive live-load shear, the beam being loaded to the *right* of the section. (2) The *minimum shear* at any section to the left of the center may be found by combining the dead-load shear with the maximum negative live-load shear, the beam being loaded to the *left* of the section. Near the end of the beam, these minimum shears will be positive, but near the center, where the live-load negative shears exceed numerically the dead-load positive shears, the resulting values will be negative. For sections on the right of the center, exactly the reverse of this holds true: the shears are numerically equal to, but of opposite sign from, those on the left of the center.

56. In the design of beams and plate girders, the sign of the shear is usually immaterial, and the maximum numerical value is all that is needed. In the case of trusses, however, it is essential to know the sign of the shear and of the stresses resulting therefrom. Furthermore, in many cases it will be necessary to find not only the maximum shear and stress but also the minimum values, so that the greatest range of stress to which the member is subjected may be known.

MAXIMUM MOMENTS AND SHEARS IN TRUSSES

57. *Notation—Reactions.*—So far, it has been assumed that the load was applied at every point along the beam. In the case of trusses and plate-girder bridges with floor systems, the load is applied along the stringers and transmitted to the trusses or girders by the floorbeams. A truss, for example, is subjected to loads acting at the joints of the chords. It is necessary to find the maximum moments

at the joints and the maximum shears in the panels. In what follows, l will denote the length of the span; p , the length of one panel; n , the number of panels; w' , the dead load per unit of length; w'' , the live load per unit of length; W' , the panel dead load; W'' , the panel live load; R_1 , the left reaction; and R_n , the right reaction. When necessary to distinguish between reactions due to the dead and to the live load, the former will be denoted by one accent— R_1' , R_n' ; the latter, by two— R_1'' , R_n'' .

58. The loads W' , W'' are taken as acting at the joints. By referring to Fig. 25, where $n = 6$, it will be observed that the number of panel points (the end joints are not counted)

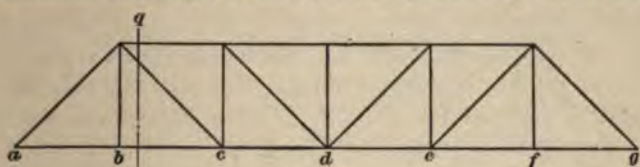


FIG. 25

is 5, or $6 - 1$. In general, the number of loaded joints is $n - 1$, or one less than the number of panels. This being the case, the total dead load on the truss is $(n - 1) W'$, and the total live load, when all the truss is loaded, is $(n - 1) W''$. Therefore, in this case,

$$R_1' = \frac{1}{2}(n - 1) W' = \frac{1}{2}(n - 1) p w'$$

$$R_1'' = \frac{1}{2}(n - 1) W'' = \frac{1}{2}(n - 1) p w''$$

It should be kept in mind that w' and w'' are, in this discussion, loads per unit of length for *one truss*, not for the *bridge*.

59. Dead-Load Moments and Shears.—The dead-load moments and shears are determined as for a beam loaded at various points. In the case of a truss, those points are the panel points, each of which carries a load equal to W' . Thus, the moment at b , Fig. 25, is $R_1' p$; at c , $R_1' \times 2p - W' p$; at d , $R_1' \times 3p - W' \times 2p - W' p$; etc. The shear between a and b is R_1' ; between b and c , $R_1' - W'$; between c and d , $R_1' - 2W'$; etc. The shear between a and b

is referred to as the shear in the panel ab ; that between b and c , as the shear in the panel bc ; etc.

60. Live-Load Moments.—The maximum live-load bending moment at any joint occurs when the whole truss is loaded; that is, when a load equal to W'' is applied at every joint of the loaded chord; and is calculated in the same manner as the dead-load moment. Thus, the maximum live-load bending moment at d is equal to

$$R_1'' \times 3p - W'' \times 2p - W'' \times p$$

61. Live-Load Shears.—For the maximum positive live-load shear at any section in a beam, it has been shown that the beam should be fully loaded up to that section from the right, and should have no load on the left of the section. In any panel bc of the truss represented in Fig. 25, the positive shear on any section q equals the left reaction minus the load, if any, at the joint b . For a maximum positive shear, the live load should extend up to joint c at least, for up to this point the reaction is the only force acting on the left of the section, and this increases as the load is moved from the right until it reaches c . If the load advances beyond c , the joint b will begin to receive some load, and the shear will then be equal to the left reaction minus the load

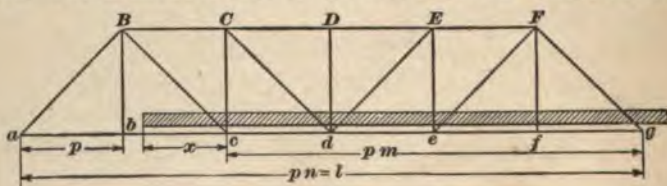


FIG. 26

at b . As the load continues to move to the left, the reaction is increased, but the load at b is also increased, so that the shear in the panel may be increased or decreased by such movement, according to the relative increase in the left reaction and the load at b . There is a certain point in each panel to which the load should extend in order that the shear in that panel may be a maximum. That point is determined as explained in the next article.

62. Let bc , Fig. 26, be any panel in which the maximum live-load shear is to be determined. Let the load, whose amount per unit of length will be denoted by w'' , cover all the panels on the right of c , and extend a distance x to the left. The number of panels in the truss will be denoted by n , and the number of panels on the right of c , by m .

It can be shown that the greatest shear in bc occurs when

$$x = \frac{pm}{n-1} \quad (1)$$

and that the value of the shear, for one truss, is then given by the formula

$$V'' = W'' \frac{m^2}{2(n-1)} \quad (2)$$

NOTE.—The derivation of formulas 1 and 2 is as follows: For the reaction R_1'' , found by the usual methods, we have

$$R_1'' = \frac{w''(x+pm)^2}{2l}$$

The load on the left of c is $w''x$, whose lever arm about c is $\frac{x}{2}$. This load is to be resolved into two components, one acting at b and the other at c . Let W_b'' be the component at b , or the partial panel load at the latter point. Taking moments about c ,

$$W_b'' p = w''x \times \frac{x}{2} = \frac{w''x^2}{2};$$

whence

$$W_b'' = \frac{w''x^2}{2p}$$

The live-load shear V_{bc}'' in the panel bc is equal to $R_1'' - W_b''$; or, substituting the values of R_1'' and W_b'' ,

$$\begin{aligned} V_{bc}'' &= \frac{w''(x+pm)^2}{2l} - \frac{w''x^2}{2p} \\ &= \frac{w''}{2} \times \frac{px^2 + 2p^2mx + p^3m^2 - lx^2}{lp} \\ &= \frac{w''}{2} \times \frac{p^3m^2 + 2p^2mx - (l-p)x^2}{lp} \end{aligned}$$

or, since $l = np$, and, therefore, $l-p = np - p = (n-1)p$,

$$\begin{aligned} V_{bc}'' &= \frac{w''}{2} \times \frac{p^3m^2 + 2p^2mx - (n-1)px^2}{lp} \\ &= \frac{w''}{2} \times \frac{p^3m^2 + 2p^2mx - (n-1)x^2}{l} \\ &= \frac{w''}{2} \times \frac{p^3m^2 + (n-1)\left(\frac{2pm}{n-1}x - x^2\right)}{l} \\ &= \frac{w''}{2} \times \frac{p^3m^2 + (n-1)\left[\left(\frac{pm}{n-1}\right)^2 - \left(\frac{pm}{n-1} - x\right)^2\right]}{l} \end{aligned}$$

This expression evidently has its greatest value when $\frac{p m}{n-1} - x = 0$. Hence, for the position of maximum live-load shear,

$$x = \frac{p m}{n-1},$$

which is formula 1.

Making $\left(\frac{p m}{n-1} - x\right)^2 = 0$, and writing $p n$ for l , the resulting value of the maximum shear becomes

$$\begin{aligned} V_{bc}'' &= \frac{w''}{2} \times \frac{p^2 m^2 + (n-1) \frac{p^2 m^2}{(n-1)^2}}{\frac{p n}{n-1}} \\ &= \frac{w''}{2} \times \frac{p m^2 + \frac{p m^2}{n-1}}{\frac{n}{n-1}} \\ &= \frac{w''}{2} \times \frac{p m^2 (n-1) + p m^2}{n(n-1)} = \frac{w'' p m^2}{2(n-1)} \end{aligned}$$

or, since $w'' p$ is the value W'' of one panel load,

$$\text{Max. } V_{bc}'' = W'' \frac{m^2}{2(n-1)},$$

which is formula 2.

EXAMPLE.—Referring to Fig. 26, suppose that the panel length is 15 feet, and that the live load is 1,200 pounds per linear foot. Required: (a) the distance x that the load should extend to the left of c , in order that the live-load shear in the panel bc may be a maximum; (b) the value of that maximum shear for one truss.

SOLUTION.—(a) To apply formula 1, we have $p = 15$, $m = 4$, and $n = 6$. Substituting in the formula,

$$x = \frac{15 \times 4}{6-1} = 12 \text{ ft. Ans.}$$

(b) To apply formula 2, we have, in addition to the values just stated,

$$W'' = \frac{1,200 \times 15}{2} = 9,000 \text{ lb.}$$

Substituting in the formula,

$$V_{bc}'' = 9,000 \left(\frac{4^2}{2(6-1)} \right) = 14,400 \text{ lb. Ans.}$$

EXAMPLES FOR PRACTICE

1. A five-panel bridge whose panel length is 14 feet carries a live load of 1,440 pounds per foot. Required: (a) the distance x that the load should extend to the left of the third joint (counted from the left end of the bridge) in order that the live-load shear in that panel may be a maximum; (b) the value of the maximum shear in that panel, for one truss.

Ans. $\begin{cases} (a) 10.5 \text{ ft.} \\ (b) 11,340 \text{ lb.} \end{cases}$

2. A seven-panel bridge whose panel length is 16 feet carries a live load of 1,500 pounds per foot. Required: (a) the distance x that the load should extend to the left of the fourth joint (counted from the left end of the bridge) in order that the live-load shear in that panel may be a maximum; (b) the value of the maximum shear in that panel, for one truss.

$$\text{Ans. } \begin{cases} (a) 10\frac{2}{3} \text{ ft.} \\ (b) 16,000 \text{ lb.} \end{cases}$$

3. (a) In the bridge in example 2, how far should the load extend to the left of the third joint (counted from the right end of the bridge) in order that the live-load shear in that panel should be a maximum? (b) What is the value of that shear for one truss?

$$\text{Ans. } \begin{cases} (a) 5\frac{1}{3} \text{ ft.} \\ (b) 4,000 \text{ lb.} \end{cases}$$

63. Approximate Method of Calculation.—In calculating the maximum live-load shear in any panel, it is customary to assume that all panel points on one side of the panel are loaded with one full-panel load and that those on the other side have no load. Thus, the maximum live-load shear in bc , Fig. 26, is found by assuming each of the joints f , e , d , and c , to carry a full-panel load, and the joint b to be unloaded. The shear bc is then taken equal to the left reaction corresponding to this assumed distribution of the load. It is, however, obviously impossible fully to load one panel point of a truss by means of a uniform load placed on the floor, without also causing a half-panel load at the next panel point. For example, the point c cannot be fully loaded unless the floor in the panels bc and cd is loaded, and this gives a half-panel load at the point b . If the load at b is neglected, and it is assumed that there is a full-panel load at c , the computed shear in the panel bc will be too large. This approximate method, therefore, gives results larger than the actual shears, but, as the error is not great, it is usually neglected. The error is greatest near the center of the truss, and zero at the end—a result not altogether undesirable, as it tends to increase the size of the small web members near the center.

GENERAL METHODS OF CALCULATING STRESSES

INTRODUCTION

64. Conditions of Equilibrium.—The stresses in the members of a structure are determined by applying the general conditions of equilibrium established and illustrated in *Analytic Statics*, Part 2, and in *Graphic Statics*. As there stated, when a structure or any part of it is in equilibrium under the action of forces, the forces fulfil the following conditions: (1) the algebraic sum of their vertical components is equal to zero; (2) the algebraic sum of their horizontal components is equal to zero; (3) the algebraic sum of their moments about any point or line is equal to zero.

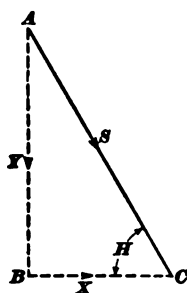


FIG. 27

If S , Fig. 27, represents the stress in any member; H , the angle that the member makes with a horizontal line; X , the horizontal component of S ; and Y , the vertical component; then, any two of the quantities S, X, Y can be found from the right triangle ABC when the other quantity and the angle H are known. Thus,
 $X = S \cos H$, $Y = S \sin H = X \tan H$, $S = X \sec H$, etc.

The functions of the angle H can usually be determined from the dimensions of the truss, in which the lengths of the horizontal and of the vertical members are generally known. Thus, for calculating the stresses in Bc , Fig. 29 (a), we have

$$\tan H = \tan Bc b = \frac{Bb}{bc};$$

$$\sec H = \frac{Bc}{bc} = \frac{\sqrt{Bb^2 + bc^2}}{bc} = \sqrt{1 + \left(\frac{Bb}{bc}\right)^2}, \text{ etc.}$$

As usual, forces acting upwards will be treated as positive, and those acting downwards as negative.

For a vertical member, the angle H is 90° , the horizontal component is zero, and the vertical component is equal to the stress in the member; for a horizontal member, the angle H is zero, the horizontal component is equal to the stress in the member, and the vertical component is zero.

As explained in *Analytic Statics*, Part 2, the three conditions of equilibrium are thus expressed algebraically:

$$\sum X = \sum S \cos H = 0$$

$$\sum Y = \sum S \sin H = 0$$

$$\sum M = 0$$

65. Free-Body Method.—In applying the foregoing principles to the calculation of the stresses in the members of a truss, the truss is considered as cut into two parts by a surface that cuts several members, among them the member or members whose stresses are sought. Either of the two parts is treated as a free body held in equilibrium by the loads, if any, that act on that part of the truss, and by external forces applied to the members cut by the surface, those external forces being numerically equal to the stresses in the corresponding members. The unknown stresses are then determined by applying the general conditions of equilibrium to the system of forces acting on that part of the truss which is treated as a free body. In order that those conditions can be applied, the cutting surface should not intersect more members in which the stresses are unknown than there are equations of equilibrium.

This subject will be fully illustrated in subsequent articles.

66. Indeterminate Stresses.—There are cases in which it is impossible to cut the truss by a surface without cutting more members than there are equations of equilibrium. The stresses in some of the members cannot then be found directly by applying the principles of statics to the members cut, and it is necessary to calculate one or more of the stresses independently, or to make an assumption regarding the distribution of stress among them. A truss

in which the stresses cannot be found by the principles of statics is said to be statically **undetermined**. The use of such trusses should be avoided as much as possible.

67. Total Stresses.—The total or resultant stress in any member is found by adding together the stress due to dead load and that due to live load. This resultant is sometimes called the **combined stress**.

68. Analytic and Graphic Methods.—Stresses may be determined either analytically or graphically. In some cases, the former method is preferable; in other cases, the latter. The analytic method gives exact results, is very easily applied to simple types of structures, such as parallel-chord trusses, and is used to a great extent in trusses that support live loads. The graphic method gives close approximations to the real stresses, is easily applicable to all types of structures, and is especially useful in trusses that support only fixed loads, particularly those trusses that have curved or inclined chords. In practice, it is customary to use the method that will give the desired result with the least amount of work. The stresses found by one method may be checked by applying the other method.

ANALYTIC METHODS

METHOD OF SECTIONS

69. General Description of the Method.—When the members cut by a surface—usually a plane—are non-

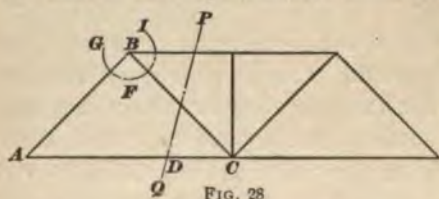


FIG. 28

concurrent, as when a plane PQ , Fig. 28, cuts two chord members and a web member, the stresses in one or two of the members cut may be found by apply-

ing the equations of equilibrium to the part on either side—customarily the left-hand side—of the cutting surface, that

part being treated as a free body. This method of determining stresses is called the **method of sections**. Some of the stresses may be determined by applying the equations of moments ($\Sigma M = 0$), and others by applying the equations of components ($\Sigma X = 0$, $\Sigma Y = 0$). The method is called also the **method of moments and shears**. When the equation of moments is used to determine the stress in one of three members cut, as AC (or AD), the origin of moments should be taken at the intersection (in this case B) of the other two.

This method, being well adapted to all cases, is used more than any other, but is of special value for the determination of horizontal and vertical stresses.

70. Special Case.—The application of the method will be understood by considering a special case. Fig. 29 (*a*) shows a truss loaded with three equal loads W . The reactions R_1 and R_2 are each equal to $\frac{3W}{2}$. It is proposed

to find the stresses in BD , Bc , and bc . The truss being cut by a plane PQ , the part on the left is treated as a free body held in equilibrium by the reaction R_1 , by the load W at b , and by the forces S_1 , S_2 , and S_3 , equal, respectively, to the stresses in BD , Bc , and bc , these forces acting at K , L , and N , as shown in Fig. 29 (*b*). The directions in which S_1 , S_2 , and S_3 act are not known, but may be assumed: if the value of a stress comes out positive, the assumed direction is correct; if negative, the stress acts in a direction opposite to the one assumed. In general, the true direction can be ascertained by inspection.

Member Bc .—The vertical forces acting on the part $aBKLN$, Fig. 29 (*b*), are R_1 , $-W$, and $-S_2 \sin H$. Therefore, the equation $\Sigma S \sin H = 0$ gives

$$R_1 - W - S_2 \sin H = 0;$$

whence
$$S_2 \sin H = R_1 - W = \frac{3W}{2} - W = \frac{W}{2}$$

and
$$S_2 = \frac{W}{2 \sin H} = \frac{W}{2} \csc H = \frac{W}{2} \frac{\sqrt{bc^2 + Bb^2}}{Bb}$$

The value of S_1 is positive; therefore, the assumed direction is correct. If the force S_1 is applied to BL at L , the joint B being supposed to be fixed, its effect will be to lengthen BL ; hence, the stress in BL , and therefore in Bc , is tension. If the value of S_1 had come out negative, this would have indicated that the direction of S_1 was opposite to

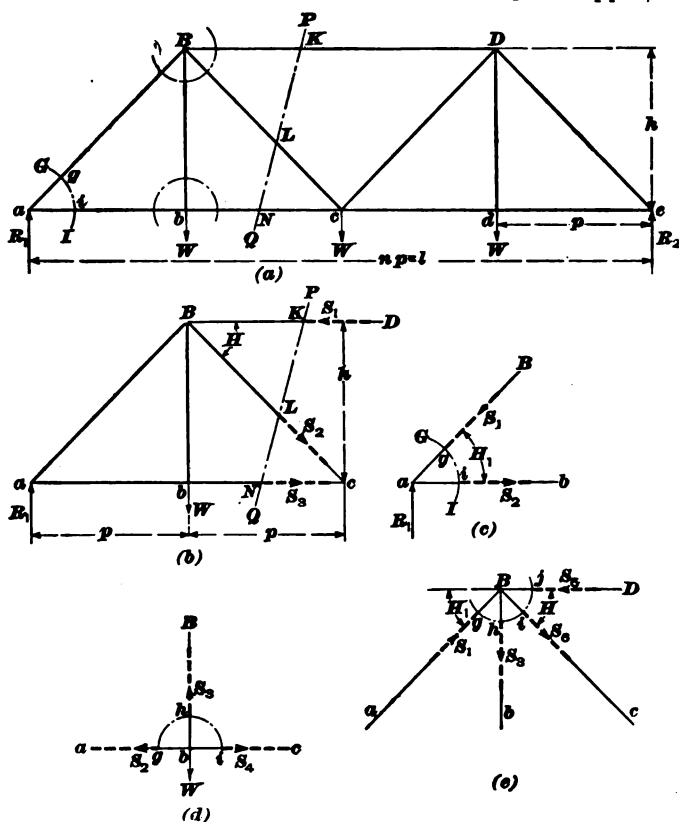


FIG. 29

that assumed, and that the stress in Bc was compression instead of tension.

It will be noticed that the vertical component of the stress in Bc is the shear on the truss at the section PQ . For this reason, the method of determining stresses by applying the

equation $\sum S \sin H = 0$ is called the **method of shears**. This method is especially useful in finding web stresses.

Member BD.—From the equation $\sum M = 0$, taking the point c as the center of moments and assuming that S_1 acts toward the point B , we have, since the lever arms and, therefore, the moments of S_2 and S_3 about c are each zero,

$$R_1 \times 2p - W \times p - S_1 \times h = 0;$$

whence

$$S_1 = \frac{R_1 \times 2p - W \times p}{h} = \frac{\frac{3W}{2} \times 2p - W \times p}{h} = 2W \frac{p}{h}$$

The value of S_1 is positive, which shows that the assumed direction is correct, and that the stress in BD is compression.

Member bc.—From the equation $\sum M = 0$, taking the point B as the center of moments, and assuming that S_2 acts away from N , we have, since the lever arms and, therefore, the moments of W , S_1 , and S_3 about B are each zero,

$$R_1 \times p - S_2 \times h = 0;$$

$$\text{whence } S_2 = \frac{R_1 \times p}{h} = \frac{\frac{3W}{2} \times p}{h} = \frac{3p}{2h} W$$

The value of S_2 being positive, the assumption that S_2 acts away from the point b is correct, and the stress in bc is tension.

METHOD OF JOINTS

71. General Description of the Method.—When the truss is cut by a curved surface, as GFI , Fig. 28, in such a manner that all the members cut are concurrent, the stresses in one or two of these members can be determined by considering as a free body that part of the truss which contains the joint (in this case B) common to the members cut. This is called the **method of joints**, or the **method of resolution of forces**. The equations best adapted to this method are the equations of components $\sum S \sin H = 0$, and $\sum S \cos H = 0$.

The method of joints is especially useful for finding stresses in parallel-chord trusses that support fixed loads; it

is not well adapted to trusses with curved or inclined chords, nor to those that support live loads.

72. Illustrative Example.—Let it be required to calculate the stresses in the members of the truss shown in Fig. 29 (*a*). Since there are, in this case, only two equations of equilibrium, it is necessary to begin with the joint *a*, where only two stresses—that in *ab* and that in *aB*—are unknown. The truss is cut by the surface *GI*, and the part containing the joint *a* is treated as a free body, as shown in Fig. 29 (*c*). The forces acting on this part are R_1 , S_1 , and S_2 , the last two being numerically equal to the stresses in *aB* and *ab*, respectively. In this case, H_1 is the angle made by *aB* with the horizontal. The functions of H_1 (or of *Bab*) can be found from the triangle *Bab*, Fig. 29 (*a*).

The vertical component of S_1 is $-S_1 \sin H_1$; that of S_2 is 0; therefore, the equation $\Sigma Y = 0$ gives

$$R_1 - S_1 \sin H_1 = 0,$$

whence

$$S_1 = \frac{R_1}{\sin H_1} = R_1 \csc H_1$$

The horizontal component of S_1 is equal to

$$-S_1 \cos H_1 = -\frac{R_1}{\sin H_1} \cos H_1 = -R_1 \cot H_1$$

The equation $\Sigma X = 0$ gives, since the reaction R_1 , being vertical, has no *X* component,

$$S_2 - R_1 \cot H_1 = 0;$$

whence

$$S_2 = R_1 \cot H_1$$

It is necessary to consider joint *b* next before going to joint *B*. The part containing the joint *b* is shown in Fig. 29 (*d*) as a free body. The forces acting are S_2 , already determined, W , S_3 , and S_4 . As neither S_2 nor W has a horizontal component, the forces S_3 and S_4 must be numerically equal; and, as neither S_2 nor S_4 has a vertical component, S_3 must be numerically equal to W . This determines the stresses in *bB* and *bc*.

The joint *B* is shown in Fig. 29 (*e*) as a free body acted on by the forces S_1 and S_2 , already determined, and by the unknown forces S_5 and S_6 , which are equal, respectively, to

the stresses in BD and Bc . Since the force S_1 , being horizontal, has no vertical component, the equation $\Sigma Y = 0$ takes the following form:

$$S_1 \sin H_1 - S_2 - S_3 \sin H_1 = 0;$$

whence

$$S_3 = \frac{S_1 \sin H_1 - S_2}{\sin H_1}$$

The horizontal component of S_3 can now be found, and putting the algebraic sum of this component, that of S_1 , and that of S_2 (which is equal to S_3 itself) equal to zero, the value of S_2 can be found.

GRAPHIC METHODS

73. Method of Moments and Shears.—In the graphic method, the moments and shears are found by means of the force polygon and the funicular, or equilibrium polygon. The stresses are then found in the same way as in the analytic method of sections. This graphic method (of moments and shears) is as a rule no shorter than the analytic method for finding the stresses in bridge trusses, and its use is not recommended for the ordinary cases of loading. There are special cases of loading, which will be discussed later, in which it is sometimes employed.

Fig. 30 (*a*) shows a truss with five panel loads W . The force polygon for the external forces, including the reactions, is shown in Fig. 30 (*c*), and the equilibrium polygon is shown in Fig. 30 (*b*). As explained in *Graphic Statics*, the moment at any point, such as c , is equal to the intercept $c'c''$ multiplied by the normal ray N . The shear in any panel, such as bc , is equal to $R_1 - W$, which in the force polygon is $(0-1) - (1-2) = 0-2$.

When the truss is cut by the plane PQ , Fig. 30 (*a*), the moment of the stress in BC about c is numerically equal to the moment of the external forces on one side of PQ about c (see Art. 70), and therefore the stress in BC is obtained by dividing that moment by the lever arm Cc , which is the depth of the truss. The vertical component in Bc is equal to *minus* the shear in the panel bc ; that is, to *minus* the algebraic sum of the external forces on the left of PQ . This

follows from the fact that, if V is the shear on PQ and Y is the vertical component of the stress in BC , then

$$Y + V = 0; \text{ whence } Y = -V$$

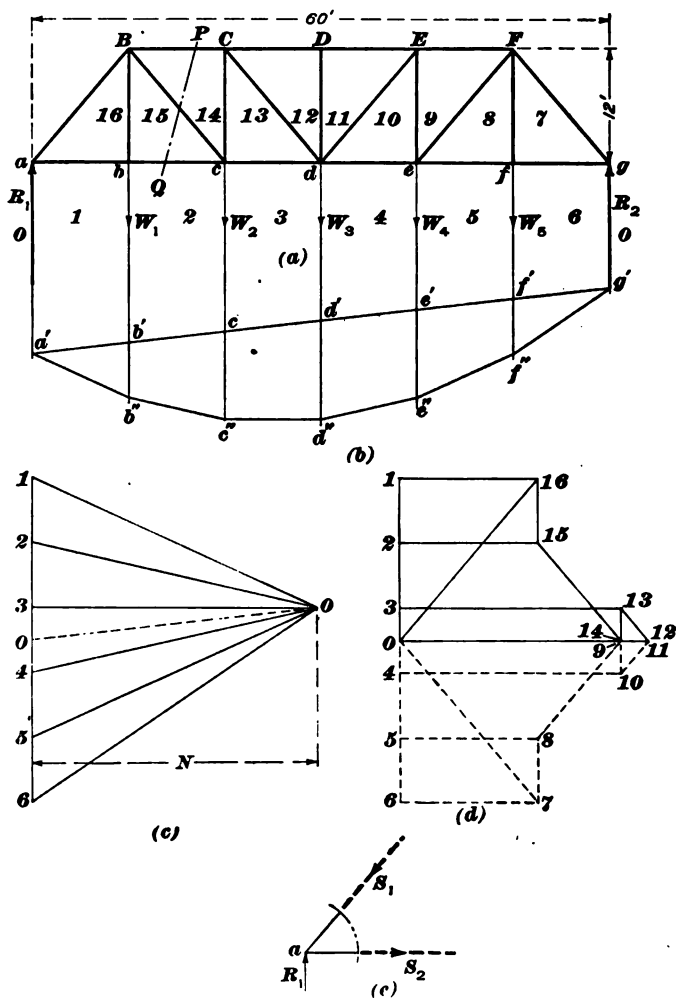


FIG. 30

74. Method by the Stress Diagram.-- The stresses in the members may also be found by the stress diagram in the

same way as by the method of joints. The stress diagram is first drawn for a joint at which the number of unknown stresses does not exceed two, then for the next joint at which the number of unknown stresses does not exceed two, and so on until the desired stresses are found. This is called the **method by the stress diagram**, and is very well adapted to trusses that support a fixed load, especially those having curved or inclined chords. When the graphic method is shorter than the analytic, the stress diagram gives the result with less work than the graphic method of moments and shears, and is, therefore, preferable.

Fig. 30 (*d*) is the stress diagram for the truss shown in Fig. 30 (*a*). The portion of the diagram in full lines gives the stresses in all the members to the left of the center; the portion in dotted lines gives the stresses in all the members to the right of the center, and is drawn simply as a check, the truss and loading being symmetrical.

The character of the stress in any member may be found from the sense of the vector that represents the stress in the member. As explained in *Graphic Statics*, if the sense of one of the vectors of a force polygon representing a balanced system is known, the senses of all the others may be found by taking them in cyclic order with the known vector. For example, at joint *a*, Fig. 30 (*a*), there are three forces, as shown in Fig. 30 (*e*): R_1 , S_1 , and S_2 . The sense of R_1 is upwards and is represented by the vector $0-1$ in the stress diagram; the sense of S_2 is given by taking $1-16$, and of S_1 by taking $16-0$ in cyclic order with $0-1$. Then, S_2 is a pull, as the direction of $1-16$ is toward the right, away from joint *a*; and S_1 is a push, as the direction of $16-0$ is downwards to the left toward joint *a*. Therefore, the stress in ab is tension, and that in aB is compression. In like manner, the character of the stress in any other member may be determined.

EXAMPLES FOR PRACTICE

1. Using the analytic method of sections, calculate the stresses in the members bc , bC , and BC of the truss shown in Fig. 31, assuming

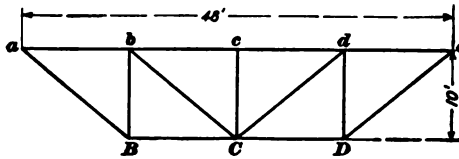


FIG. 31

a load of 5,000 pounds at each of the joints b , c , and d .

$$\text{Ans. } \begin{cases} \text{Stress in } bc = 12,000 \text{ lb. compression} \\ \text{Stress in } bC = 3,900 \text{ lb. tension} \\ \text{Stress in } BC = 9,000 \text{ lb. tension} \end{cases}$$

2. Using the analytic method of joints, calculate the stresses in the members that meet at joint b of the truss shown in Fig. 32.

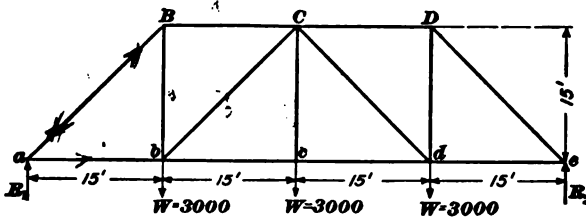


FIG. 32

$$\text{Ans. } \begin{cases} \text{Stress in } ab = 4,500 \text{ lb. tension} \\ \text{Stress in } Bb = 4,500 \text{ lb. tension} \\ \text{Stress in } bC = 2,120 \text{ lb. compression} \\ \text{Stress in } bc = 6,000 \text{ lb. tension} \end{cases}$$

3. Using the stress diagram, determine the stresses in all the members of the left half of the truss shown in Fig. 32, assuming a load of 4,500 pounds at each of the joints b , c , and d .

$$\text{Ans. } \begin{cases} \text{Stress in } aB = 9,550 \text{ lb. compression} \\ \text{Stress in } ab = 6,750 \text{ lb. tension} \\ \text{Stress in } Bb = 6,750 \text{ lb. tension} \\ \text{Stress in } BC = 6,750 \text{ lb. compression} \\ \text{Stress in } bC = 3,180 \text{ lb. compression} \\ \text{Stress in } bc = 9,000 \text{ lb. tension} \\ \text{Stress in } Cc = 4,500 \text{ lb. tension} \end{cases}$$

STRESSES IN BRIDGE TRUSSES

(PART 2)

PARALLEL-CHORD TRUSSES

THE SINGLE-SYSTEM WARREN TRUSS

INTRODUCTION

1. **Description.**—The Warren truss, Fig. 1, is a simple type of truss with parallel chords, in which the web members are all inclined and make the same angle with the vertical, giving the truss the appearance of a series of connected isosceles triangles; it is sometimes called the *triangular*

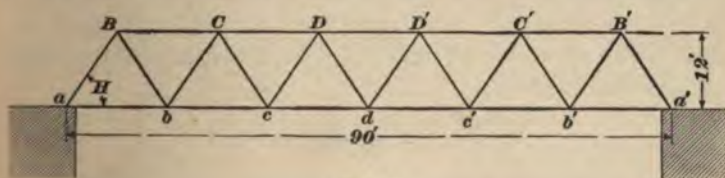


FIG. 1

truss. The Warren truss is used in deck, through, and half-through bridges, is more frequently built as a riveted than as a pin-connected truss, and is especially adapted to the shorter spans for which trusses are used. For spans up to about 100 feet, it is frequently spoken of as a *lattice girder*. For longer spans, it is sometimes built with subdivided panels, or with multiple systems of web members.

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2. Methods of Calculation.—The stresses in the members of the simple type of Warren truss can be readily found, either graphically or analytically, by applying the general conditions of equilibrium. The work of calculation by the analytic methods is so simple that the graphic method is seldom used in practice for this type of truss.

The analytic methods are illustrated in the following articles, which contain the calculations of the maximum and minimum stresses in all the members of the six-panel truss shown in Fig. 1. This truss has a span of 90 feet and a height of 12 feet; the dead load is taken as 600 pounds, and the live load as 1,600 pounds, per linear foot of the bridge; all the dead load is assumed to be applied at the joints of the loaded chord, and the truss is assumed to support one-half the entire load on the bridge.

METHOD OF SECTIONS

3. Panel Loads and Reactions.—The dead panel load W' for one truss is equal to $\frac{600}{2} \times 15 = 4,500$ pounds.

As explained in *Stresses in Bridge Trusses*, Part 1, the number of panel loads considered in determining the reactions is one less than the number of panels in the truss. In this case, the number of panels in the truss is six; therefore, only five panel loads are taken into account in determining the reactions. The reactions R_1' and R_6' , Fig. 2 (a), due to the dead load are each equal to

$$\frac{4,500 \times 5}{2} = 11,250 \text{ pounds}$$

The live panel load W'' for one truss is equal to $\frac{1,600}{2} \times 15 = 12,000$ pounds; and the reactions R_1'' and R_6'' for a fully loaded truss are each equal to

$$\frac{12,000 \times 5}{2} = 30,000 \text{ pounds}$$

4. Chord Stresses in General.—Chord stresses may be conveniently determined by the method of sections

explained in *Analytic Statics*, Part 2, and in *Stresses in Bridge Trusses*, Part 1. The dead loads and reactions are shown in Fig. 2 (a). The method will be illustrated by determining the dead-load stresses in CD and cd . The truss may be considered cut by a plane q intersecting the members CD , cD , and cd . The portion of the truss to the left of section q is shown in Fig. 2 (b), the external forces being R_1' at a ,

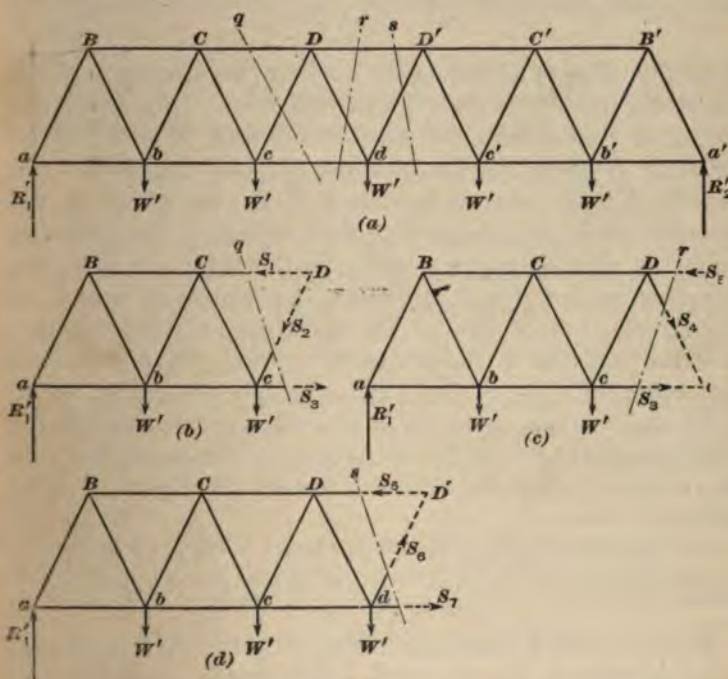


FIG. 2

W' at b , W' at c , and the forces S_1 , S_2 , and S_3 , equal numerically to the stresses in the members cut.

Assuming that the stress in CD is compression, S_1 will be directed toward the left, and its magnitude may be computed by taking moments of all the forces about c , the point of intersection of S_2 and S_3 . The moments of the load at c and of S_2 and S_3 are each equal to zero, since their lever arms are zero. Writing the equation for the

moments of all the forces shown in Fig. 2 (*b*) about the point *c*, we have

$$R_1' \times 30 - W' \times 15 - S_1 \times 12 = 0;$$

whence

$$\begin{aligned} S_1 &= \frac{R_1' \times 30 - W' \times 15}{12} = \frac{\frac{5W'}{2} \times 30 - W' \times 15}{12} \\ &= \frac{4W' \times 15}{12} \end{aligned}$$

As the value of S_1 comes out positive, the assumption that the stress in CD is compression is correct. The advantage of taking *c* as a center of moments is that, by so doing, an equation is obtained that contains but one unknown force, namely, S_1 ; the other two, S_2 and S_3 , do not appear in the equation, since the moment of each is zero. The center of moments may be taken at any point, whether it is on the structure or not, but it is better, if possible, to take it at the intersection of two of the members, thereby eliminating the stresses in those members from the equation of moments.

To find S_2 , the center of moments may be taken at the intersection *D* of S_1 and S_3 . Assuming the stress in cd to be tension, S_2 will be directed toward the right. Taking moments about *D*,

$$R_1' \times 37.5 - W' \times 22.5 - W' \times 7.5 - S_2 \times 12 = 0;$$

$$\text{whence } S_2 = \frac{R_1' \times 37.5 - W' \times 22.5 - W' \times 7.5}{12}$$

If this comes out positive, the assumption that the stress in cd is tension is correct; if negative, the stress is compression, but its numerical value will be that determined by the last equation.

In a similar manner, if the stress in DD' is required, the truss may be considered cut by a plane *r*, intersecting DD' , Dd , and cd , or by a plane *s*, intersecting DD' , dD' , and dd' . The portion to the left of section *r* is shown in Fig. 2 (*c*); the portion to the left of section *s* is shown in Fig. 2 (*d*). The proper center of moments is *d*. The stress in DD' will be assumed as compression; then, S_2 will be

directed toward the left. Writing the expression for the moment at d ,

$$R_1' \times 45 - W' \times 30 - W' \times 15 - S_2 \times 12 = 0;$$

$$\text{whence } S_2 = \frac{R_1' \times 45 - W' \times 30 - W' \times 15}{12}$$

In Fig. 2 (c), the lever arms of S_2 and S_3 are each zero; in Fig. 2 (d), the lever arms of S_2 and S_3 and of the load at d are each zero; hence, these do not appear in the equation of moments.

The values of the other chord stresses can be found in a similar manner. All upper-chord members will be in compression and all lower-chord members in tension.

5. It will be seen that the numerator of the expression for the stress in any chord member is, in each case, the sum of the moments of the panel loads and reactions at the left of the section, about the joint opposite the member. This sum is the bending moment on the truss at that point. The denominator is the height of the truss. We may, therefore, state the following general principle:

The stress in any chord member of a simple Warren truss is equal to the bending moment on the truss, at the joint opposite the member considered, divided by the height of the truss.

6. Dead-Load Chord Stresses.—Applying the principle just stated to the determination of the dead-load chord stresses, the following values are found:

$$\text{Stress in } a b = \frac{\text{moment at } B}{\text{height}} = \frac{11,250 \times 7.5}{12} \\ = 7,030 \text{ pounds, tension.}$$

$$\text{Stress in } b c = \frac{\text{moment at } C}{\text{height}} = \frac{11,250 \times 22.5 - 4,500 \times 7.5}{12} \\ = 18,280 \text{ pounds, tension.}$$

$$\text{Stress in } c d = \frac{\text{moment at } D}{\text{height}} \\ = \frac{11,250 \times 37.5 - 4,500 \times 22.5 - 4,500 \times 7.5}{12} \\ = 23,910 \text{ pounds, tension.}$$

$$\text{Stress in } BC = \frac{\text{moment at } b}{\text{height}} = \frac{11,250 \times 15}{12}$$

$$= 14,060 \text{ pounds, compression.}$$

$$\text{Stress in } CD = \frac{\text{moment at } c}{\text{height}} = \frac{11,250 \times 30 - 4,500 \times 15}{12}$$

$$= 22,500 \text{ pounds, compression.}$$

$$\text{Stress in } DD' = \frac{\text{moment at } d}{\text{height}}$$

$$= \frac{11,250 \times 45 - 4,500 \times 30 - 4,500 \times 15}{12}$$

$$= 25,310 \text{ pounds, compression.}$$

As the truss is symmetrical, the stresses in the members on the right of the center are equal to those in the corresponding members on the left. That is, the stress in $D'C'$ is equal to the stress in CD ; the stress in $C'B'$ is equal to the stress in BC ; etc.

7. Live-Load Chord Stresses.—The maximum bending moments, and, therefore, the maximum chord stresses, due to a moving load, occur when the truss is fully loaded. This condition of loading is similar to the dead loading, each panel load being now 12,000 pounds, and each reaction 30,000 pounds, in place of 4,500 and 11,250 pounds, respectively. The chord stresses may be found in precisely the same way as for dead loads; thus,

$$\text{stress in } ab = \frac{30,000 \times 7.5}{12} = 18,750 \text{ pounds}$$

and so on. The results are (using the minus sign for tension and the plus sign for compression):

MEMBER	STRESS, IN POUNDS
ab	− 18,750
bc	− 48,750
cd	− 63,750
BC	+ 37,500
CD	+ 60,000
DD'	+ 67,500

8. As all the dead load is assumed as being applied at the joints of the loaded chord, the live-load stresses in the

chords may be obtained from the dead-load stresses by multiplying the latter by the ratio of the live to the dead load per linear foot, which ratio is $\frac{1,600}{600}$, or $\frac{8}{3}$. For example, the

dead-load stress in bc is $-18,280$, and the live-load stress is

$$-18,280 \times \frac{8}{3} = -48,750 \text{ pounds}$$

9. Maximum and Minimum Chord Stresses.—Since the live-load stress in any chord member is of the same sign as the dead-load stress, the maximum stress in the member is equal to the sum of the two stresses; and the minimum stress is equal to the dead-load stress.

10. Web Stresses in General.—The stresses in the web members may be found by the method of shears, explained in *Stresses in Bridge Trusses*, Part I. For example, to determine the stress in the web member cD , Fig. 2 (*a*), the portion of the structure to the left of section q may be considered as a free body, as shown in Fig. 2 (*b*). Any of the conditions of equilibrium may be applied to the forces shown. It is desirable, if possible, to use an equation that contains the web force S_s , to be determined, but which does not involve either of the two forces S_i and S_a . As S_i and S_a are horizontal, they will not appear in the equation $\Sigma Y = \Sigma S \sin H = 0$. Assuming the stress in cD to be compression, S_s will act downwards to the left. Writing the expression for $\Sigma Y = \Sigma S \sin H = 0$ gives

$$\Sigma Y = R'_1 - W' - W' - \text{vertical component of } S_s = 0;$$

that is, denoting the angle DcS_s by H ,

$$R'_1 - 2W' - S_s \sin H = 0;$$

whence

$$S_s \sin H = R'_1 - 2W'$$

and

$$S_s = \frac{R'_1 - 2W'}{\sin H} = (R'_1 - 2W') \csc H$$

The term $R'_1 - 2W'$ is the shear on the section q ; therefore, the vertical component $S_s \sin H$ of S_s is numerically equal to the shear on the plane of section that cuts cD . In general, the following principle may be stated:

For single-system parallel-chord trusses, the vertical component of the stress in any web member is numerically equal to the shear

on the plane of section cutting that web member and the two chord members between which the web member lies; and the stress in the same web member is numerically equal to the shear just referred to, multiplied by the cosecant of the angle that the member makes with the horizontal.

11. Character of Web Stresses.—If the shear on section q , Fig. 2 (b), is positive, the resultant of the external vertical forces on the left of the section acts upwards; then S_2 must act downwards, and the stress in cD is compression. If the shear is negative, S_2 acts upwards, and the stress in cD is tension. If the shear on section r , Fig. 2 (c), is positive, the resultant of the external forces on the left acts upwards, S_4 acts downwards, and the stress in Dd is tension. If the shear is negative, S_4 acts upwards, and the stress in Dd is compression. These conclusions may be stated as a general principle thus:

In those web members inclining downwards toward the left or upwards toward the right, positive shear causes compression, and negative shear tension; in those web members inclining upwards toward the left or downwards toward the right, positive shear causes tension, and negative shear compression.

12. Dead-Load Shears and Web Stresses.—In order to calculate the stresses in the web members due to dead load, it will be convenient first to find the shears on the sections cut by the planes o, p, q' , etc., Fig. 3 (a). They are as follows:

MEMBER	SECTION	SHEAR, IN POUNDS
aB	o	+ 11,250
Bb	p	+ 11,250
bC	q'	+ 6,750
Cc	r'	+ 6,750
cD	s'	+ 2,250
Dd	t	+ 2,250
dD'	u	— 2,250

From symmetry, the shears to the right of the center d will be equal and of opposite sign to the corresponding shears on the left. For example, the shear on section

t is + 2,250 pounds, and that on section u is - 2,250 pounds. The shears on the sections o and p are equal, as are also those on q' and r' , and those on s' and t ; because, in each case, the two planes are passed between the same two panel loads, that is, in the same panel of the loaded chord; and, as in the present case there are no loads applied at the joints of the unloaded chord, the shears on all sections in any panel are equal. As all shears to the left of d are positive, the stresses in the members aB , bC , and cD that incline down-

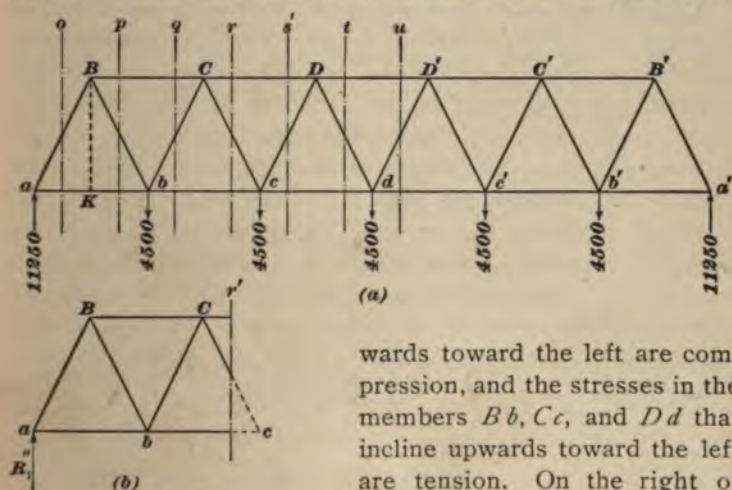


FIG. 3

wards toward the left are compression, and the stresses in the members Bb , Cc , and Dd that incline upwards toward the left are tension. On the right of the center, the shears are negative;

and the stresses in the members that incline downwards toward the left are tension, while the stresses in those that incline upwards toward the left are compression. In the present case, the stresses in aB and Bb are equal and of opposite signs, as are also those in bC and Cc , etc.

Referring to Fig. 3 (a), and applying the general principle given in Art. 10, we have

$$\begin{aligned} \csc H &= \csc BaK = \frac{Ba}{BK} = \frac{\sqrt{BK^2 + aK^2}}{BK} \\ &= \frac{\sqrt{BK^2 + \left(\frac{ab}{2}\right)^2}}{BK} = \frac{\sqrt{12^2 + 7.5^2}}{12} = 1.18 \end{aligned}$$

The web stresses can now be computed. They are as follows:

MEMBER	STRESS, IN POUNDS
$a B, a' B'$	$11,250 \times 1.18 = + 13,280$
$B b, B' b'$	$11,250 \times 1.18 = - 13,280$
$b C, b' C'$	$6,750 \times 1.18 = + 7,970$
$C c, C' c'$	$6,750 \times 1.18 = - 7,970$
$c D, c' D'$	$2,250 \times 1.18 = + 2,660$
$D d, D' d$	$2,250 \times 1.18 = - 2,660$

13. Live-Load Shears and Web Stresses.—The stresses caused in the web members by the live load may be found from the shears. As the maximum stresses are desired, the truss must be so loaded as to cause the maximum shear for each case. The approximate method of loading explained in *Stresses in Bridge Trusses*, Part 1, will be used. The maximum positive shear in any panel occurs when all joints to the right of the panel are loaded; the maximum negative shear occurs when all joints to the left are loaded. Thus, in member Cc , Fig. 3 (*a*), the maximum tension occurs when all joints from c to b' are loaded; and the maximum compression occurs when the joint b is loaded. When joints c to b' are loaded, the left reaction is

$$\frac{12,000 \times (1 + 2 + 3 + 4)}{6} = 20,000 \text{ pounds}$$

As there is no load at b , the only force acting on the portion of the truss to the left of r' is the left reaction. Then, the shear in the panel bc is equal to the left reaction, Fig. 3 (*b*), or 20,000 pounds. The stress in Cc is equal to the shear in panel bc multiplied by $\csc H$; or,

$$\text{stress in } Cc = 20,000 \times 1.18 = 23,600 \text{ pounds, tension}$$

In like manner, the stress in any other member may be found. The maximum positive live shears are as follows:

PANEL	LOAD	SHEAR, IN POUNDS
$a b$	From b to b'	30,000
$b c$	From c to b'	20,000
$c d$	From d to b'	12,000
$d c'$	At c' and b'	6,000
$c' b'$	At b'	2,000

In the panel $b'a'$ there can be no positive shear.

The maximum negative shear in any panel is numerically equal to the maximum positive shear in the corresponding panel at the other end of the truss. The maximum and minimum stresses in the members can now be found by multiplying the respective shears by $\csc H$. These stresses are given in the following table:

Panel	Member	Positive Shear Pounds	Stress Due to Positive Shear Pounds	Negative Shear Pounds	Stress Due to Negative Shear Pounds
ab	aB	30,000	+ 35,400		
ab	Bb	30,000	- 35,400		
bc	bC	20,000	+ 23,600	2,000	- 2,360
bc	Cc	20,000	- 23,600	2,000	+ 2,360
cd	cD	12,000	+ 14,160	6,000	- 7,080
cd	Dd	12,000	- 14,160	6,000	+ 7,080

14. Combined Shears and Web Stresses.—The maximum and minimum stresses caused in the members on the left of the center by combined dead and live loads may be found by multiplying the maximum and minimum shears, respectively, by $\csc H$. The maximum shear in any panel is equal to the sum of the positive dead-load and the positive live-load shear in the panel; the minimum shear is equal to the algebraic sum of the positive dead-load and the negative live-load shear in the panel. In columns 3 and 5 of the following table are given the maximum and minimum shears, respectively; while in columns 4 and 6 are given the maximum and minimum stresses, respectively, each stress being obtained by multiplying the corresponding shear by $\csc H$.

In the members aB and Bb , the minimum stresses are equal to the dead-load stresses, as there can be no negative live-load shear in the panel ab . The minimum stresses in cD and Dd are of opposite sign to the maximum, because

in the panel cd the negative live-load shear exceeds the positive dead-load shear. Under the special conditions here assumed, the combined shear is positive when the joints to the right of d are loaded, and negative when those to the

1	2	3	4	5	6
Panel	Member	Maximum Shear Pounds	Maximum Stress Pounds	Minimum Shear Pounds	Minimum Stress Pounds
ab	aB	+ 41,250	+ 48,680	+ 11,250	+ 13,280
ab	Bb	+ 41,250	- 48,680	+ 11,250	- 13,280
bc	bC	+ 26,750	+ 31,570	+ 4,750	+ 5,610
bc	Cc	+ 26,750	- 31,570	+ 4,750	- 5,610
cd	cD	+ 14,250	+ 16,820	- 3,750	- 4,420
cd	Dd	+ 14,250	- 16,820	- 3,750	+ 4,420

left of d are loaded. This is an important point, and shows that, in the present case, the members cD and Dd are sometimes in tension and sometimes in compression, according to the position of the live load. The two values of the stress given for each member are the extreme values that can

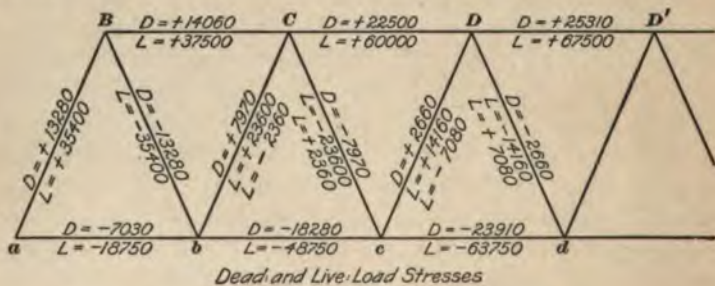
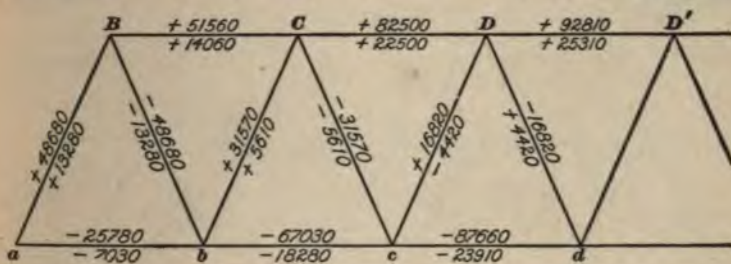


FIG. 4

exist in that member for the given loads. In each of the members aB , Bb , bC , and Cc , the stress may have any value between the extreme values, and such stresses will always be of the same kind, that is, tension or compression. In each of the members cD and Dd , the stress may have

any value between the positive and the negative value given. The stress in each member will reverse when the combined shear changes from positive to negative.

The stresses in all the members are shown in Figs. 4 and 5, which should be carefully studied. In Fig. 4, L represents



Maximum and Minimum Stresses

FIG. 5

the live-load stress, and D the dead-load stress. In Fig. 5, the maximum is placed above and the minimum below the line representing the member. Notice how the maximum and minimum stresses in Fig. 5 are obtained by addition from the stresses in Fig. 4.

METHOD OF JOINTS

15. For purposes of comparison, the maximum and minimum stresses in the example of Art. 2 will be calculated by the method of joints. The truss is represented in Fig. 6 (A), the dead panel load being 4,500 pounds, and the live panel load, 12,000 pounds. The figure gives

$$\cot H = \cot B a K = \frac{a K}{B K} = \frac{7.5}{12} = .625$$

and, as before (Art. 12), $\csc H = 1.18$.

16. **Dead-Load Stresses.**—It will be convenient to start at joint a , as there are only two unknown stresses at that joint.

Joint a .—This joint is represented as a free body in Fig. 6 (a), the forces acting on it being the reaction $R_1' = 11,250$ pounds, and the forces S_1 and S_2 , the last two

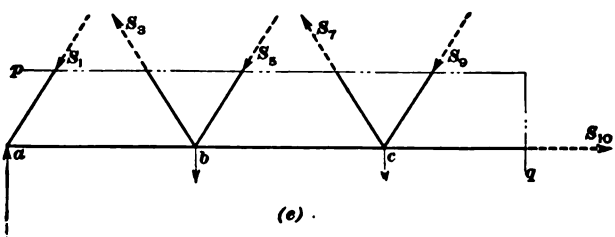
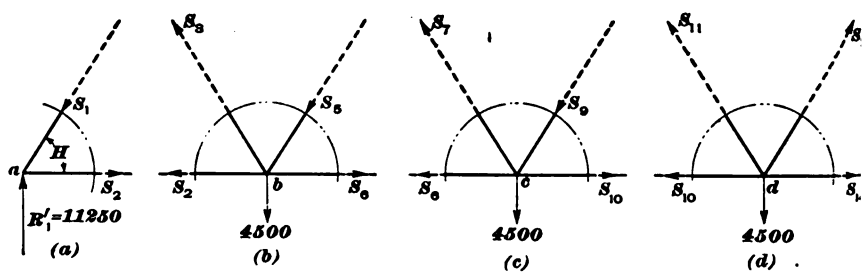
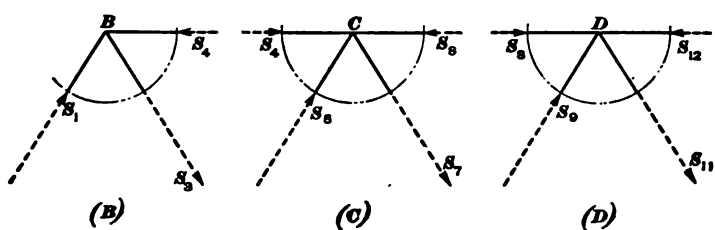
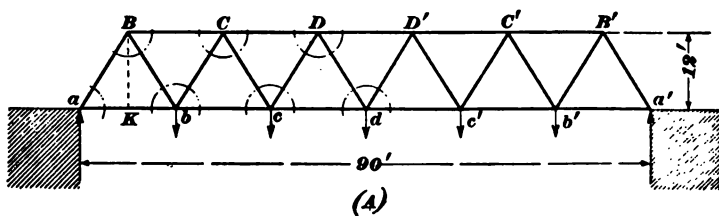


FIG. 6

representing the stresses in aB and ab , respectively. The stress in aB will be assumed as compression, and that in ab as tension. Then, S_1 will act downwards to the left, and S_2 horizontally to the right. From the general conditions of equilibrium, we have, since the vertical component of S_2 is zero,

$$\Sigma Y = R_1' - S_1 \sin H = 0;$$

whence $S_1 \sin H = R_1' = 11,250$ pounds

and

$$\begin{aligned} S_1 &= 11,250 \csc H = 11,250 \times 1.18 \\ &= 13,280 \text{ pounds, compression in } aB \end{aligned}$$

Likewise, since R_1' has no horizontal component,

$$\Sigma X = S_1 \cos H - S_2 = 0;$$

whence

$$\begin{aligned} S_2 &= S_1 \cos H = 11,250 \csc H \cos H \\ &= 11,250 \frac{\cos H}{\sin H} \\ &= 11,250 \cot H = 11,250 \times .625 \\ &= 7,030 \text{ pounds, tension in } ab \end{aligned}$$

Joint B.—This joint is represented in Fig. 6 (B), the forces acting on it being S_1 , S_2 , and S_3 , which represent the stresses in aB , Bb , and BC , respectively. The force S_1 is known and acts upwards to the right; the stress in Bb will be assumed as tension, and that in BC as compression. Then, S_2 will act downwards to the right, and S_3 horizontally to the left. From the conditions of equilibrium,

$$\Sigma Y = S_1 \sin H - S_2 \sin H = 0;$$

whence $S_2 \sin H = S_1 \sin H = 11,250$ pounds

and $S_2 = 11,250 \csc H = 13,280$ pounds, tension in Bb

Likewise,

$$\Sigma X = S_1 \cos H + S_2 \cos H - S_3 = 0;$$

whence

$$\begin{aligned} S_3 &= (S_1 + S_2) \cos H = (11,250 \csc H + 11,250 \csc H) \cos H \\ &= 22,500 \csc H \cos H = 22,500 \cot H \\ &= 14,060 \text{ pounds, compression in } BC \end{aligned}$$

Joint b.—This joint is represented in Fig. 6 (b), the forces acting on it being S_2 , S_3 , S_4 , and S_5 , which represent the stresses in ab , Bb , bC , and bc , respectively; and the panel

load of 4,500 pounds. The latter acts vertically downwards, S_2 acts horizontally to the left, and S_3 acts upwards to the left; S_4 and S_5 are unknown. The stress in bC will be assumed as compression, and that in bc as tension. Then, S_4 will act downwards to the left and S_5 horizontally to the right.

$$\Sigma Y = S_3 \sin H - 4,500 - S_4 \sin H = 0;$$

whence

$$S_3 \sin H = S_4 \sin H - 4,500 = 11,250 - 4,500 \\ = 6,750 \text{ pounds}$$

and

$$S_3 = 6,750 \csc H = 7,970 \text{ pounds, compression in } bC$$

Likewise,

$$\Sigma X = S_4 + S_3 \cos H + S_5 \cos H - S_6 = 0;$$

whence

$$S_6 = S_4 + S_3 \cos H + S_5 \cos H \\ = 11,250 \cot H + 11,250 \csc H \cos H + 6,750 \csc H \cos H \\ = 29,250 \cot H = 18,280 \text{ pounds, tension in } bc$$

Joint C.—This joint is represented in Fig. 6 (*C*), the forces acting on it being S_4 , S_5 , S_7 , and S_8 , which represent the stresses in BC , bC , Cc , and CD , respectively; S_4 acts horizontally to the right, S_5 upwards toward the right, while S_7 and S_8 are unknown. The stress in Cc will be assumed as tension, and that in CD as compression. Then, S_7 will act downwards to the right, and S_8 horizontally to the left.

$$\Sigma Y = S_5 \sin H - S_7 \sin H = 0;$$

whence

$$S_7 \sin H = S_5 \sin H = 6,750 \text{ pounds}$$

and

$$S_7 = 6,750 \csc H = 7,970 \text{ pounds, tension in } Cc$$

Likewise,

$$\Sigma X = S_4 + S_5 \cos H + S_7 \cos H - S_8 = 0;$$

whence

$$S_8 = S_4 + S_5 \cos H + S_7 \cos H \\ = 22,500 \cot H + 6,750 \csc H \cos H + 6,750 \csc H \cos H \\ = 36,000 \cot H = 22,500 \text{ pounds, compression in } CD$$

Joint c.—This joint is represented in Fig. 6 (*c*), the forces acting on it being S_6 , S_7 , S_9 , and S_{10} , which represent the

stresses in bc , Cc , cD , and cd , respectively; S_a acts horizontally to the left, and S_r upwards to the left; while S_s and S_{10} are unknown. The stress in cD will be assumed as compression, and that in cd as tension. Then, S_s will act downwards to the left, and S_{10} horizontally to the right.

$$\Sigma Y = S_r \sin H - S_s \sin H - 4,500 = 0;$$

whence

$$S_r \sin H = S_s \sin H - 4,500 = 6,750 - 4,500 = 2,250 \text{ pounds}$$

and

$$S_s = 2,250 \csc H = 2,660 \text{ pounds, compression in } cD$$

Likewise,

$$\Sigma X = S_a + S_r \cos H + S_s \cos H - S_{10} = 0;$$

whence

$$\begin{aligned} S_{10} &= S_a + S_r \cos H + S_s \cos H \\ &= 29,250 \cot H + 6,750 \csc H \cos H + 2,250 \csc H \cos H \\ &= 38,250 \cot H = 23,910 \text{ pounds, tension in } cd \end{aligned}$$

Joint D.—This joint is represented in Fig. 6 (*D*), the forces acting on it being S_a , S_s , S_{11} , and S_{12} , which represent the stresses in CD , cD , Dd , and DD' , respectively; S_a acts horizontally to the right, and S_s upwards to the right; while S_{11} and S_{12} are unknown. The stress in Dd will be assumed as tension, and that in DD' as compression. Then, S_{11} will act downwards to the right, and S_{12} horizontally to the left.

$$\Sigma Y = S_s \sin H - S_{11} \sin H = 0;$$

whence

$$S_{11} \sin H = S_s \sin H = 2,250 \text{ pounds}$$

and

$$S_{11} = 2,250 \csc H = 2,660 \text{ pounds, tension in } Dd$$

Likewise,

$$\Sigma X = S_a + S_s \cos H + S_{11} \cos H - S_{12} = 0;$$

whence

$$\begin{aligned} S_{12} &= S_a + S_s \cos H + S_{11} \cos H \\ &= 36,000 \cot H + 2,250 \csc H \cos H + 2,250 \csc H \cos H \\ &= 40,500 \cot H = 25,310 \text{ pounds, compression in } DD' \end{aligned}$$

Joint d.—This joint is represented in Fig. 6 (*d*). It is evident at once that S_{12} is numerically equal to S_{11} , and that S_{12} is numerically equal to S_{10} . Therefore, the stress in dD' is 2,660 pounds tension, and that in $d'c'$ is 23,910 pounds tension.

It is unnecessary to proceed further than joint d , as the stresses in the members at the right end are the same as those in the corresponding members at the left end.

From the preceding discussion, it may be seen that the stress in any web member is equal to the algebraic sum of all the vertical forces that act on the truss on the left of the member considered, that is, to the shear in the panel in which the member is located, multiplied by $\csc H$ (see Art. 10).

13. 17. For the chord members, it is convenient to refer again to the stress in one of the members, such as cd , Fig. 6 (c) (joint c). The equation $\sum X = 0$ gives

$$S_{10} = S_6 + S_7 \cos H + S_8 \cos H$$

Substituting for S_6 its value $S_2 + S_3 \cos H + S_4 \cos H$,

$$S_{10} = S_2 + S_3 \cos H + S_4 \cos H + S_7 \cos H + S_8 \cos H$$

Likewise, substituting for S_8 its value $S_1 \cos H$,

$$S_{10} = S_1 \cos H + S_2 \cos H + S_3 \cos H + S_7 \cos H + S_4 \cos H$$

Letting Y_1, Y_2 , etc. represent the vertical components of the stresses S_1, S_2 , etc., and substituting for S_1, S_2 , etc. their values $Y_1 \csc H, Y_2 \csc H$, etc., respectively, we have

$$\begin{aligned} S_{10} &= Y_1 \csc H \cos H + Y_2 \csc H \cos H + Y_3 \csc H \cos H \\ &\quad + Y_7 \csc H \cos H + Y_4 \csc H \cos H \\ &= Y_1 \cot H + Y_2 \cot H + Y_3 \cot H + Y_7 \cot H \\ &\quad + Y_4 \cot H = (Y_1 + Y_2 + Y_3 + Y_7 + Y_4) \cot H \end{aligned}$$

Now, $Y_1 \cot H, Y_2 \cot H$, etc. are the horizontal components of the stresses in aB, Bb , etc., respectively, and the sum of these components from $Y_1 \cot H$ to $Y_4 \cot H$ is the algebraic sum of the horizontal components of the stresses in all the web members that connect with the lower chord at the left of cd . In like manner, it may be shown that the stress in DD' is equal to the algebraic sum of the horizontal components of the stresses in all the web members that connect with the upper chord to the left of DD' . In general,

The stress in any portion of either chord is equal to the algebraic sum of the horizontal components of the stresses in all the web members that connect with the chord at the left of the portion considered.

18. In the present case, the web members all make the same angle with the horizontal; that is, H is constant, and the stress S_{10} in a chord member, such as cd , is equal to $(Y_1 + Y_2 + Y_3 + Y_4 + Y_5) \cot H$. Letting ΣY represent the sum of the vertical components in all the web members that connect with a chord at the left of any portion, then, the stress in that portion may be found by the formula

$$S = \Sigma Y \times \cot H$$

In applying this formula to the determination of the stress in a chord member, care must be taken that the horizontal components are given the proper signs. For example, for cd , the truss may be considered cut by the section pq , Fig. 6 (e), and the portion below and to the left of this section treated as a free body. The horizontal forces that act on this portion are the horizontal components of S_1 , S_2 , etc., and the stress in cd : S_1 , S_2 , and S_3 act downwards to the left, and S_4 and S_5 act upwards to the left; therefore, all the horizontal components of these stresses act to the left, and in finding S_{10} the vertical components of the stresses from S_1 to S_5 must be *added* numerically to find ΣY .

From the foregoing, the following general rule is derived:

To find the stress in any web member of a single-system Warren truss by the method of joints, multiply the shear in the panel in which the member is located by $\csc H$; to find the stress in any chord member, multiply by $\cot H$ the algebraic sum of all the shears used in obtaining the stresses in all the web members that connect with the chord at the left of the member considered.

The application of this rule can be greatly simplified by constructing a diagram, as shown in Fig. 7. A sketch of the truss is drawn (not necessarily to scale) and on the upper side of each web member is written, with its proper sign and as the coefficient of $\csc H$, the shear in the panel in which the member is located. On the upper side of each chord member is written, as the coefficient of $\cot H$, the algebraic sum of the shears that have been written on all the web members that connect with the chord at the left of the member considered. On the under side of each member is written the stress obtained by performing the indicated multiplication.

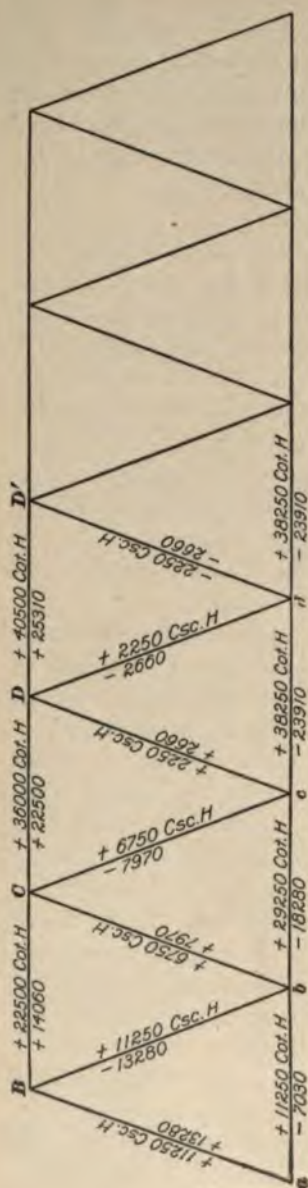


FIG. 7



FIG. 8

The plus and minus signs written before the stresses indicate, as usual, compression and tension, respectively. Thus, in Fig. 7, the coefficient $+6,750$ of $\csc H$ on cC is the shear in the panel bc ; the product $6,750 \csc H$ gives $7,970$, which is the numerical value of the stress in cC . The coefficient $+29,250$ of $\cot H$ on bc is the algebraic sum of the shears written on the members ab , Bb , and bC ; the product $29,250 \cot H$ gives $18,280$, which is the numerical value of the stress in bc .

19. Live-Load Chord Stresses.—As the chord stresses are greatest when the truss is fully loaded, it is necessary first to find the shears due to a full live load. They are as follows:

PANEL	SHEAR, IN POUNDS
ab	30,000
bc	18,000
cd	6,000

These values are written on the upper side of the web members, as shown in Fig. 8, and the coefficients of $\cot H$ for the chord stresses are found by adding the shears as explained in Art. 18. Then, the stresses in the chord members are obtained by performing the multiplications indicated, and the results written on the under sides of the members.

20. Live-Load Web Stresses.—The shears that were found in Art. 19 are those due to full live load, the shear in any panel being the difference between the left reaction and the sum of all the panel loads between the left reaction and the panel under consideration. For example, the shear in panel cd due to full live load is

$$\frac{12,000 (1 + 2 + 3 + 4 + 5)}{6} - (12,000 + 12,000)$$

or,

$$\begin{aligned} & \left[\frac{12,000 (1 + 2 + 3)}{6} \right] + \left[\frac{12,000 (4 + 5)}{6} - (12,000 + 12,000) \right] \\ &= \left[\frac{12,000 (1 + 2 + 3)}{6} \right] - \left[\frac{12,000 (1 + 2)}{6} \right] \end{aligned}$$

The expression contained in the left-hand brackets of the last member of this equation is the left reaction that would be caused by loads at d , c' , and b' , if they were the only loads on the truss; it was explained in Art. 13 that this is the maximum positive live-load shear in panel cd . Also, the expression contained in the right-hand bracket is the maximum negative live-load shear in panel cd . From this the following principle is obtained:

The shear in any panel of a truss due to full live load is equal to the algebraic sum of the maximum live-load positive shear and the maximum live-load negative shear that can occur in that panel.

Let V'' = shear in any panel due to full live load;

V_p'' = maximum positive live-load shear that can occur in that panel;

V_n'' = maximum negative live-load shear that can occur;

then,

$$V'' = V_p'' + V_n'';$$

whence

$$V_p'' = V'' - V_n'';$$

that is, the maximum positive live-load shear in any panel may be found by subtracting algebraically the maximum negative live-load shear that can occur in the panel from the shear due to full live load. This principle is of special value in finding live-load web stresses by the method of joints; the shear in each panel due to full load is found in connection with the chord stresses; the maximum negative shear in each panel is then found in order to get the minimum stresses in the members; then, the maximum positive shear in any panel may be found by subtracting algebraically the maximum negative shear from the shear in the panel due to full load.

In panel ab there can be no negative shear. Then, in this panel the maximum live-load stresses occur when the truss is fully loaded; $\csc H$ may be written after the shear that has been written on aB and Bc (30,000 pounds), Fig. 8, and the stresses found by multiplying that shear by $\csc H$ (1.18). The results are written on the other side of the lines that represent aB and Bb . On the other web members,

directly under the values of the shear due to full load, are written, as coefficients of $\csc H$, the maximum negative live-load shear and the maximum positive live-load shear, the latter being obtained by subtracting, algebraically, the negative shear from the shear due to full load. The maximum and minimum stresses are obtained by performing the multiplications indicated, and the results are written on the under side of the members. As stated in Art. 18, the plus and minus signs written before the stresses represent compression and tension, respectively. Thus, in panel $b c$, Fig. 8, the shear due to full load (+18,000 pounds) is written on $b C$ and $C c$; the maximum negative shear (-2,000 pounds) is written under +18,000, and is the coefficient of $\csc H$ for the minimum live-load stresses in $b C$ and $C c$. The algebraic difference between the shear due to full load and the maximum negative shear, $+18,000 - (-2,000) = +20,000$ pounds, is then written under -2,000 $\csc H$, as the coefficient of $\csc H$, for the maximum live-load stresses in $b C$ and $C c$. These stresses are obtained by performing the multiplications, and their values are written on the under sides of the members. Thus, for the member $b C$, the maximum live-load stress is $20,000 \csc H$, or +23,600 pounds; the minimum live-load stress is $-2,000 \csc H$, or -2,360 pounds.

From the foregoing, the following general rule is obtained:

To find the maximum and minimum live-load stresses in the web members of a single-system Warren truss, when the shears due to full live load are known, write on each member the maximum negative live-load shear in the panel in which the member is located, and multiply it by $\csc H$ for the minimum stress; subtract, algebraically, the maximum negative shear from that due to full load, and multiply the result for each member by $\csc H$ for the maximum stresses.

The combined stresses are found in the same way as in Art. 14.

EXAMPLE.—The truss represented in Fig. 9 is a seven-panel through Warren truss, with dimensions as shown. The dead load is 1,000 pounds, and the live load, 2,000 pounds per linear foot of bridge. Assuming that one-third of a dead panel load is applied at each of the

joints of the upper chord, and two-thirds at the joints of the lower chord, find, by the method of joints, the maximum and minimum stresses in all the members.

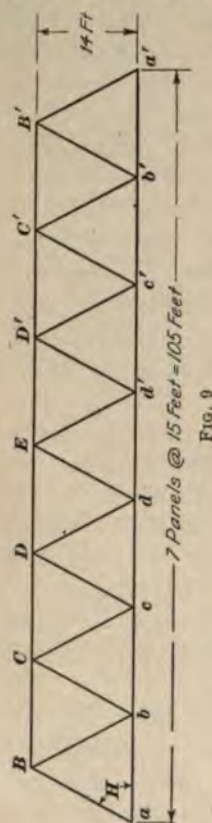


FIG. 9

SOLUTION.—Each dead panel load is equal to

$$\frac{1,000 \times 15}{2} = 7,500 \text{ pounds}$$

of which 5,000 pounds is applied at the joints of the lower chord, and 2,500 pounds at the joints of the upper chord. It will be noticed that there are six joints in the lower chord and seven in the upper chord. It is customary to assume that one-third of a panel load is applied at each of the joints of the upper chord. The dead-load reaction for one truss is equal to

$$\frac{5,000 \times 6}{2} + \frac{2,500 \times 7}{2} = 23,750 \text{ pounds}$$

Each live panel load is equal to

$$\frac{2,000 \times 15}{2} = 15,000 \text{ pounds}$$

and the live-load reaction for one truss fully loaded is equal to

$$\frac{15,000 \times 6}{2} = 45,000 \text{ pounds}$$

As a portion of the dead load is applied at the upper-chord joints, which lie midway between the joints of the lower chord, the dead-load shear in any panel of the lower chord is not constant. For example, in the panel $b c$, the dead-load shear from b to C is equal to

$$23,750 - (5,000 + 2,500) = 16,250 \text{ pounds}$$

while from C to c it is

$$23,750 - (5,000 + 2,500 + 2,500), \text{ or } 13,750 \text{ pounds}$$

The figure gives

$$\csc H = \frac{\sqrt{14^2 + 7.5^2}}{14} = 1.134; \cot H = \frac{7.5}{14} = .5357$$

The dead-load stresses, found by the rule given in Art. 18, are indicated in Fig. 10 (a); the live-load stresses, found by the rule given in Art. 20, are indicated in Fig. 10 (b). As the dead-load and live-load stresses are not required separately, the work will be shortened in the present case by combining the coefficients of $\csc H$ and $\cot H$, respectively, and indicating the maximum and minimum stresses, as represented in Fig. 10 (c). There remains now simply the operation of multiplying these coefficients by $\csc H$ and $\cot H$, respectively, to get the maximum and the minimum combined stresses, as represented in Fig. 10 (d). The student should verify the values of these

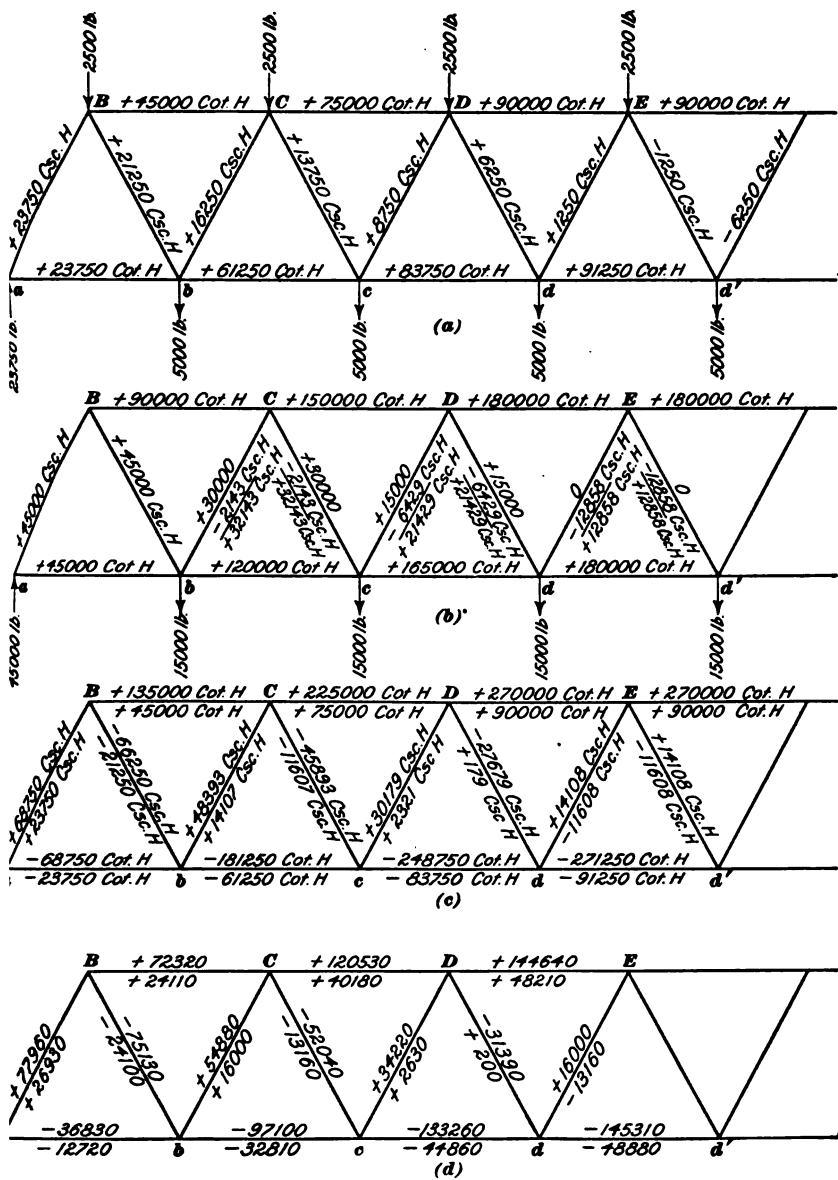


FIG. 10

stresses. The signs of the coefficients given in Figs. 10 (a) and (b) are the signs of the shears; the minus and plus signs in Fig. 10 (c) and (d) represent tension and compression, respectively.

THE DECK WARREN TRUSS

21. When used in a deck bridge, the Warren truss may be supported either as shown in Fig. 11 or as shown in Fig. 12. The live load is supported at the joints of the upper chord; the dead load may be assumed to be applied at

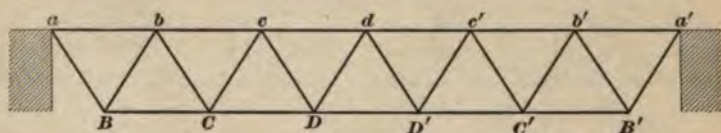


FIG. 11

the joints of the upper chord, or one-third of it at the joints of the lower chord. In calculating the stresses, the same methods and rules are used as for the through truss.

In Fig. 11, each panel load is a full load. In Fig. 12, the loads are supported between A and B , and between B' and A' ,

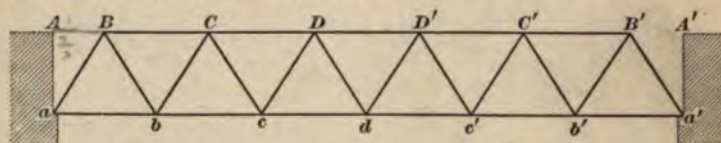


FIG. 12

by the end stringers, one end of which rests on the abutments, and the other end connects with the floorbeam at B or B' . As the distances AB and $B'A'$ are each equal to a half panel, each of the joints B and B' supports three-quarters of a panel load, and this value must be used at these joints in the calculation of reactions and stresses.

EXAMPLES FOR PRACTICE

1. Suppose that the truss represented in Fig. 9 has a span of 112 feet, and a height of 16 feet; if the dead load is equal to 800 pounds, all of which is applied at the joints of the loaded chord, and the live load is 1,800 pounds per linear foot of bridge, find: (a) the maximum and minimum combined stresses in the members bc , CD , and $d d'$, using

the method of joints; (b) the maximum and minimum combined stresses in the members aB , bC , and dD , using the method of sections.

		MEMBER	STRESS, IN POUNDS	
			MAXIMUM	MINIMUM
Ans.	(a)	$b c$	- 83,200	- 25,600
		$C D$	+ 104,000	+ 32,000
		$d d'$	- 124,800	- 38,400
	(b)	$a B$	+ 69,800	+ 21,500
		$b C$	+ 48,900	+ 12,000
		$d D$	- 30,200	- 250

2. Let Fig. 9 represent a seven-panel deck Warren truss having the same loads and dimensions as in example 1 and supported in a manner similar to that shown in Fig. 12. What are the maximum and minimum stresses due to combined dead and live load: (a) in the members bC , CD , and dd' , using the method of sections? (b) in the members Bb , bC , and Dd , using the method of joints?

		MEMBER	STRESS, IN POUNDS	
			MAXIMUM	MINIMUM
Ans.	(a)	bC	- 85,800	- 26,400
		CD	+ 101,400	+ 31,200
		dd'	- 127,400	- 39,200
	(b)	Bb	- 59,000	- 17,000
		bC	- 59,000	- 17,000
		Dd	- 21,700	+ 6,500

THE WARREN TRUSS WITH SUBVERTICALS

22. Description.—The simple type of Warren truss can be used for span lengths up to about 125 feet. For longer spans, it is impossible to fulfil the economical conditions of height, panel length, and slope of diagonals. If the proper height of truss is used and the diagonals are given an economical inclination, the panels will be too long, and it is advisable to subdivide them. This may be accomplished in several ways, one of which is to use a Warren truss with vertical members attached to the joints of the unloaded chord, dividing each panel of the loaded chord into two equal panels. The truss is then called the **Warren truss with subverticals**. The vertical members are tension members in a through truss and compression members in a deck truss. All the other members correspond in every way to those in the through Warren truss in Fig. 1. The method of calculation is the same as for the single-system Warren truss.

In Fig. 13 (a) is represented a twelve-panel through Warren truss with subverticals, having a span of 180 feet and a height of 24 feet. Each panel load will be denoted by W , and the reactions, as usual, by R_1 and R_2 .

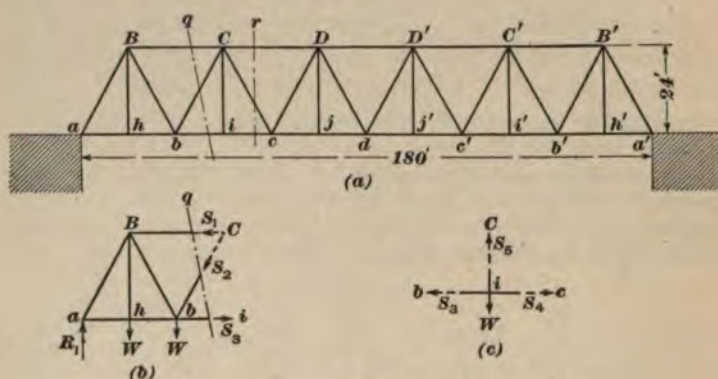


FIG. 13

23. Chord Stresses.—The stresses in the upper-chord members may be found by dividing the bending moments at the opposite joints b, c, d , etc. by the height of the truss; the stresses in the lower-chord members may be found by means of the bending moments at B, C, D , etc. (see Art. 5). Thus, for the stresses in BC and bi , the truss may be considered cut by a plane q . The portion to the left of section q is shown in Fig. 13 (b), S_1 , S_2 , and S_3 representing the stresses in the members BC , bi , and bc , respectively. The stress in BC is compression, and so S_1 will act horizontally to the left; the stress in bi is tension, and so S_2 will act horizontally to the right. For the stress in BC , the center of moments is taken at b . Then,

$$\sum M = R_1 \times 30 - W \times 15 - S_1 \times 24 = 0;$$

whence

$$S_1 = \frac{R_1 \times 30 - W \times 15}{24} = \frac{\text{bending moment at } b}{24}$$

For the stress in bi , the center of moments is taken at C . Then,

$$\sum M = R_1 \times 45 - W \times 30 - W \times 15 - S_2 \times 24 = 0;$$

whence

$$S_i = \frac{R_1 \times 45 - W \times 30 - W \times 15}{24} = \frac{\text{bending moment at } C}{24}$$

For the stress in ic , the joint i is treated as a free body, as shown in Fig. 13 (*c*). The only horizontal forces are S_i and S_c ; therefore, they are equal and opposite, and the stress in ic is equal to the stress in bi . In like manner, the stress in ah is equal to the stress in hb ; the stress in cj is equal to the stress in jd , etc. Other chord stresses may be determined in the same way as those here explained.

24. Web Stresses.—The stress in each vertical is tension and equal to the load applied at the foot of the vertical. This is evident when the equation $\Sigma Y = 0$ is applied to the forces acting on such a joint as i , Fig. 13 (*c*). The only vertical forces being S_i and the panel load W , they must be equal and opposite. Therefore, the stress in each vertical is equal to a panel load. The other web stresses may be found by the method of shears already explained (Art. 10).

25. Deck Bridge.—If the through truss in Fig. 13 is inverted and used as a deck truss, as shown in Fig. 14, the maximum stresses in members having the same letters in

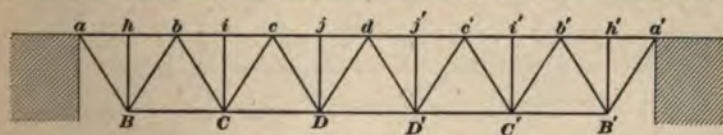


FIG. 14

the two figures will be numerically equal, but of opposite characters. If the truss is supported as shown in Fig. 15, the stresses in all the members but the verticals will be of the

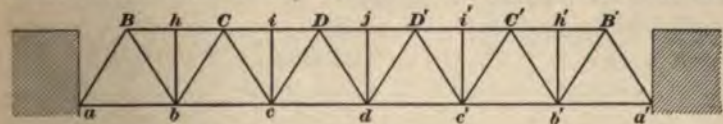


FIG. 15

same characters and have the same numerical values as those in the corresponding members in the through truss in Fig. 13. The verticals will be in compression.

EXAMPLES FOR PRACTICE

1. Suppose that, in the bridge shown in Fig. 13, the dead load is 1,000 pounds, and the live load, 2,200 pounds, per linear foot. Assume that all the dead load is applied at the joints of the loaded chord. What are the maximum and minimum stresses, due to the combined dead and live load, in all the members?

	MEMBER	STRESS, IN POUNDS	
		MAXIMUM	MINIMUM
Ans.	$aB, a'B'$	+ 155,700	+ 48,600
	$B C, B' C'$	+ 150,000	+ 46,900
	$C D, C' D'$	+ 240,000	+ 75,000
	$D D'$	+ 270,000	+ 84,400
	$a h, h b, a' h', h' b'$	- 82,500	- 25,800
	$b i, i c, b' i', i' c'$	- 202,500	- 63,300
	$c j, j d, c' j', j' d'$	- 262,500	- 82,000
	$B h, C i, D j, D' j', C' i', B' h'$	- 24,000	- 7,500
	$B b, B' b'$	- 129,000	- 38,200
	$b C, b' C'$	+ 104,000	+ 26,000
	$C c, C' c'$	- 80,500	- 12,400
	$c D, c' D'$	+ 58,700	- 2,900
	$D d, D' d$	- 38,500	+ 19,900

2. Let Fig. 16 be a ten-panel deck bridge having a span length of 150 feet and a height of 20 feet. If the dead load is 900 pounds, and the live load, 2,000 pounds, per linear foot, and it is assumed that all

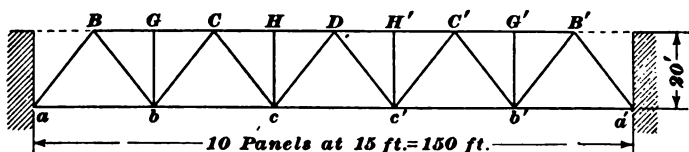


FIG. 16

the dead load is applied at the joints of the loaded chord, find: (a) the maximum and minimum stresses in the members ab , Bb , Gb , and bC due to combined dead and live load; (b) the maximum and minimum stresses in the members bc , HD , cD , and $c'c'$ due to combined dead and live load.

	MEMBER	STRESS, IN POUNDS	
		MAXIMUM	MINIMUM
Ans.	(a) ab	- 73,400	- 22,800
		- 97,000	- 27,700
		+ 21,800	+ 6,800
	(b) bc	+ 73,600	+ 15,500
		- 171,300	- 53,200
		+ 195,800	+ 60,800
	cD	+ 32,300	- 14,500
		- 203,900	- 63,300

THE DOUBLE-INTERSECTION WARREN TRUSS

26. Description.—Fig. 17 (a) shows another type of Warren truss with subdivided panels, which was extensively used in the past and is used to some extent at the present time. The simple Warren truss is shown in full lines, the panels being subdivided by the addition of the web members shown in dotted lines parallel to the full-line members and

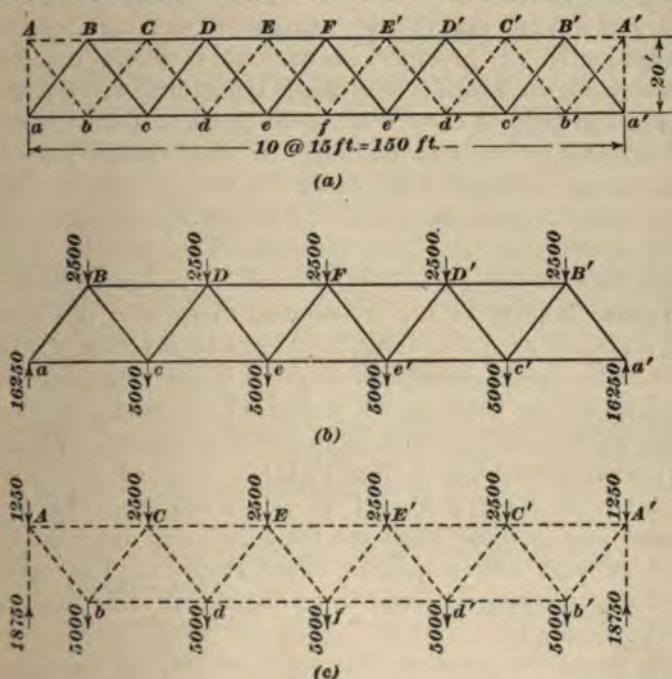


FIG. 17

half way between them. The two sets of web members are called the *two systems of web*. Such a truss is called a **double system**, or **double intersection**, Warren truss, and sometimes simply a **double Warren truss**. The joints of one system are in each case vertically opposite the joints of the other. As the end diagonals of the dotted system slope upwards, it is necessary to provide a vertical member, called the **vertical end post**, and produce the top chord at each end.

27. Methods of Calculation.—If the truss shown in Fig. 17 (*a*) is considered cut by a plane that intersects two chord members, that plane will cut also two web members, and there will be four unknown stresses. As there are only three equations of equilibrium, they are not sufficient for the determination of the four unknown stresses unless some assumption is made regarding the distribution of stress among the several members cut by the plane. It is customary to assume that the two systems of web members act independently, or, in other words, that they are the web members of two independent trusses lying in the same plane, the top and bottom chords being common to both trusses. The stresses in the web members of each system may be found from the loads that come on the system; and the chord stresses may be found by properly combining the chord stresses of the two systems. This will be made clearer by studying Figs. 17 (*b*) and (*c*). The system shown as a truss in Fig. 17 (*b*) is assumed to support the loads at *c, e, e', c', B, D, F, D',* and *B'*. The web stresses due to these loads are the actual web stresses in the corresponding members of the truss shown in Fig. 17 (*a*); and the chord stresses are partial or component chord stresses. The system shown as a truss in Fig. 17 (*c*) is assumed to support the loads at *b, d, f, d', b', A, C, E, E', C',* and *A'*. The web stresses due to these loads are the actual web stresses; the chord stresses are component chord stresses. The actual chord stresses may be found by adding the stresses found in the two systems.

The double-intersection Warren truss may be used in a deck or in a through bridge. The stresses are calculated in the same way for the two kinds. As the analytic method of calculation is shorter than the graphic, the latter will not be considered. The method of calculation can best be illustrated by an example. For this purpose, the dead-load stresses in the truss shown in Fig. 17 (*a*) will be determined. The truss has ten panels, the span length is 150 feet, and the height 20 feet. The dead load will be taken as 1,000 pounds per linear foot of bridge, one-third of which

is supposed to be applied at the unloaded chord. The method of joints is best adapted to this case.

28. Panel Loads and Reactions.—The truss may be divided into the two systems shown in Fig. 17 (*b*) and (*c*). For convenience of reference, the system shown in full lines in Fig. 17 (*b*) may be called the *primary system*, and that shown in dotted lines in Fig. 17 (*c*), the *secondary system*. The dead panel load is equal to

$$\frac{1,000 \times 15}{2} = 7,500 \text{ pounds}$$

of which 2,500 is supported at each of the top joints, and 5,000 at each of the bottom joints. The primary system supports four loads of 5,000 and five loads of 2,500 pounds. Therefore, the reaction for the primary system is

$$\frac{4 \times 5,000 + 5 \times 2,500}{2} = 16,250 \text{ pounds}$$

The secondary system supports five loads of 5,000 and four loads of 2,500 pounds. Therefore, the reaction for the secondary system is

$$\frac{5 \times 5,000 + 4 \times 2,500}{2} = 17,500 \text{ pounds}$$

In addition to this, there is a half-panel load of 1,250 pounds at each of the end joints of the top chord. Then, the total reaction for the secondary system is equal to 18,750 pounds. The loads and reactions for the primary systems are shown in Fig. 17 (*b*); those for the secondary system, in Fig. 17 (*c*). As in previous cases,

$$\cot H = \cot B a b = \frac{15}{20} = .75; \csc H = \frac{\sqrt{20^2 + 15^2}}{20} = 1.25$$

29. Web Stresses.—The stress in the vertical end post is equal to the reaction of the secondary system. The vertical components of the web stresses in each system may be written directly by finding the shears, and the stresses found from them by multiplying by $\csc H$. It should be borne in mind that each system is treated as an independent truss loaded as shown in Figs. 17 (*b*) and (*c*); also, that, in determining the shear on any section, both the lower- and the upper-chord loads should be taken into account. Thus, the shear

on a plane cutting DF , De , and ce , Fig. 17 (*b*), is $16,250 - (2,500 + 5,000 + 2,500)$, or the algebraic sum of the external forces acting at a , B , c , and D .

The web stresses, whose values should be verified by the student, are:

MEMBER	STRESS, IN POUNDS
aA	+ 18,750
aB	$16,250 \times 1.25 = + 20,300$
Ab	$17,500 \times 1.25 = - 21,900$
bC	$12,500 \times 1.25 = + 15,600$
Bc	$13,750 \times 1.25 = - 17,200$
cD	$8,750 \times 1.25 = + 10,900$
Cd	$10,000 \times 1.25 = - 12,500$
dE	$5,000 \times 1.25 = + 6,250$
De	$6,250 \times 1.25 = - 7,800$
eF	$1,250 \times 1.25 = + 1,600$
Ef	$2,500 \times 1.25 = - 3,100$

30. Chord Stresses.—As explained in Art. 17, the stress in any chord member of a single-system Warren truss is equal to the algebraic sum of the horizontal components of the stresses in all the web members that connect with the chord on the left (or right) of the member in question. For example, the stress in DF , Fig. 17 (*b*), is equal to the sum of the horizontal components in aB , Bc , cD , and De ; the stress in EE' , Fig. 17 (*c*), is equal to the sum of the horizontal components in Ab , bC , Cd , dE , and Ef . The stress in EF , Fig. 17 (*a*), equals the sum of the stresses in DF , Fig. 17 (*b*), and EE' , Fig. 17 (*c*). Therefore, the stress in EF equals the sum of the horizontal components in Ab , aB , Bc , bC , Cd , cD , De , dE , and Ef . In general,

The stress in any chord member of a double Warren truss is equal to the algebraic sum of the horizontal components of the stresses in all the web members that connect with the chord on the left (or right) of the member considered.

Keeping in mind that the horizontal component of the stress in any web member is equal to the vertical component multiplied by $\cot H$, the chord stresses may be written as follows:

MEMBER	STRESS, IN POUNDS
AB	$17,500 \times .75 = + 13,100$
BC	$(17,500 + 16,250 + 13,750) \times .75 = + 35,600$
CD	$(17,500 + 16,250 + 13,750 + 12,500 + 10,000) \times .75 = + 52,500$
DE	$(17,500 + 16,250 + 13,750 + 12,500 + 10,000 + 8,750$ $+ 6,250) \times .75 = + 63,750$
EF	$(17,500 + 16,250 + 13,750 + 12,500 + 10,000 + 8,750$ $+ 6,250 + 5,000 + 2,500) \times .75 = + 69,400$
FE'	$(17,500 + 16,250 + 13,750 + 12,500 + 10,000 + 8,750$ $+ 6,250 + 5,000 + 2,500 + 1,250 - 1,250) \times .75 = + 69,400$
ab	$16,250 \times .75 = - 12,200$
bc	$(16,250 + 17,500 + 12,500) \times .75 = - 34,700$
cd	$(46,250 + 13,750 + 8,750) \times .75 = - 51,600$
de	$(68,750 + 10,000 + 5,000) \times .75 = - 62,800$
ef	$(83,750 + 6,250 + 1,250) \times .75 = - 68,400$
fe'	$(91,250 + 2,500 - 2,500) \times .75 = - 68,400$

31. Live-Load Stresses.—The live-load stresses may be found in precisely the same way as the dead-load stresses, by separating the truss into two systems. For the maximum chord stresses, each system should be fully loaded, and the stresses in the members added together to get the combined or actual stresses. For the maximum web stresses, the portion of each system that will give the maximum shear (positive or negative) in the various panels must be loaded; the stresses found from the shears will be the actual maximum and minimum live-load stresses in the web members.

EXAMPLE FOR PRACTICE

If the live load on the bridge described in Art. 26 and illustrated in Fig. 17 is 2,200 pounds per linear foot, determine: (a) the maximum and minimum combined stresses in the members EF , Ef , eF , and ef , using the dead-load stresses found in the preceding pages; (b) the maximum and minimum combined stresses in the members BC , Bc , bC , and bc .

Ans.	MEMBER	STRESS, IN POUNDS	
		MAXIMUM	MINIMUM
(a)	EF	+ 224,100	+ 69,400
	Ef	- 21,700	+ 5,100
	eF	+ 13,900	- 10,800
	ef	- 216,900	- 68,400
	BC	+ 116,000	+ 35,600
	Bc	- 58,400	- 17,200
	bC	+ 48,600	+ 13,800
	bc	- 109,000	- 34,700

THE DOUBLE WARREN TRUSS WITH SUBVERTICALS

32. Description.—Fig. 18 (a) shows the double Warren truss with subverticals that subdivide each panel of the loaded chord into two equal panels. In this truss, the loads at the intermediate joints b, d, f , etc. act on both systems at the intersections B, D, F , etc. of the web members, and on this account it is impossible to separate the truss into two

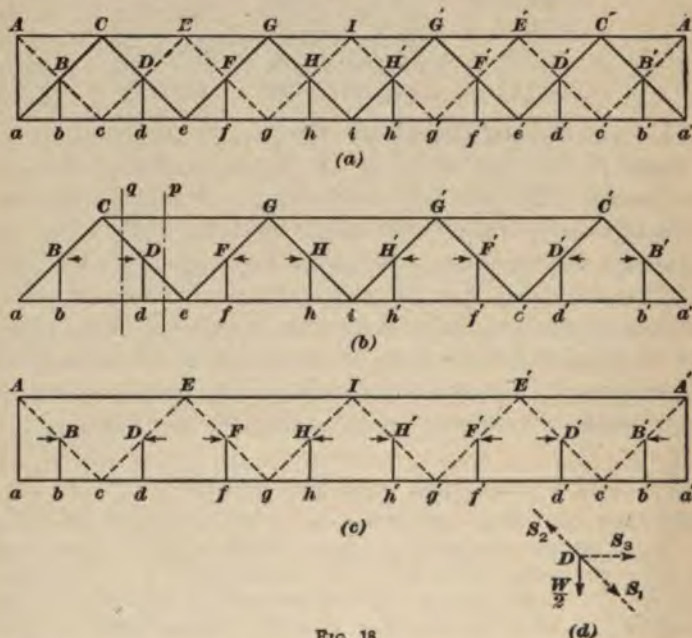


FIG. 18

independent systems. However, an assumption is usually made that is probably very close to the actual distribution of stresses. As in the case of the truss described in the preceding articles, one of the two systems formed by the inclined members is called the primary; the other, the secondary.

33. Method of Calculation.—The stress in each subvertical is evidently tension, and equal to the panel load at

its lower joint. In finding the stresses in the other members, it is customary to assume that each system carries one-half of the load transmitted by the subverticals to the joints B, D, F , etc. Thus, in Figs. 18 (b) and (c), which shows the primary and secondary systems, together with the subverticals, each of the joints b, d, f , etc. is supposed to carry one-half of a panel load to each system. The stresses are found in almost the same manner as in the double Warren truss without subverticals, except that, in treating each system as an independent truss, external forces, representing the action of the other system on the system under consideration, must be introduced at the joints D, F , etc., as will be explained presently.

34. Web Stresses.—If the general equation of equilibrium $\Sigma Y = \Sigma S \sin H = 0$ is applied to all the external forces acting on one side of a plane of section that cuts a web member and two chord members of either system, such as section q , Fig. 18 (b), it will be seen that the vertical component of the stress in the web member is equal to the shear on the section, and the stress is equal to the shear multiplied by $\csc H$. For the maximum or minimum stress in any member, the system in which the member occurs should be loaded on the right or left of the member, in the same way as in a single-system Warren truss.

Consider the sections q and p , Fig. 18 (b). Denoting the load at each lower-chord joint by W , the vertical component of the stress in CD is $R_1' - \frac{W}{2}$, and that in De is $R_1' - \frac{W}{2} - \frac{W}{2}$; then, the horizontal component of the stress in CD is equal to $\left(R_1' - \frac{W}{2}\right) \cot H$, and in De , to $\left(R_1' - \frac{W}{2} - \frac{W}{2}\right) \cot H$. Writing the expression for the sum of the horizontal forces at joint D of the primary system, shown in Fig. 18 (d), we have

$$\begin{aligned} \Sigma X &= S_2 \cos H - S_1 \cos H \\ &= \left(R_1' - \frac{W}{2}\right) \cot H - \left(R_1' - \frac{W}{2} - \frac{W}{2}\right) \cot H = \frac{W}{2} \cot H, \end{aligned}$$

from which it will be seen that there is an unbalanced force,

equal to $\frac{W}{2} \cot H$, acting horizontally to the left, which must be held in equilibrium by the force S_s , equal to $\frac{W}{2} \cot H$, acting horizontally to the right. It may be shown that at the joint D of the secondary system there is also an unbalanced force equal to $\frac{W}{2} \cot H$, acting horizontally to the left, which holds in equilibrium the unbalanced force at joint D of the primary system. This force, which may be called S_s , is exerted at each joint (B, D, F , etc.) of each system by the other system, and may be considered as an external force in writing equations.

35. Chord Stresses.—The maximum chord stresses obtain when there is a full live load; the minimum, when there is no live load on the truss. The stress in any member may be found by properly combining the partial stresses in the two systems. When all the stresses are desired, the method of joints is the shorter; when only one or two stresses are desired, the method of moments is shorter.

For example, the stress in EG , Fig. 18 (a), is equal to the stress in CG , Fig. 18 (b), plus the stress in EI , Fig. 18 (c). By the method of joints, the stress in CG , Fig. 18 (b), is equal to the sum of the horizontal components of the stresses in BC and CD ; the stress in EI is equal to the sum of the horizontal components of the stresses in AB, DE , and EF .

If it is desired to calculate the stress in any chord member by the method of moments, it is necessary to take into account the moments of the horizontal forces S_s . As these forces are alternately opposite in direction, it is convenient to pass the planes of section through the truss in such a way that there will be an even number of intermediate joints on the portion of the truss considered. Then, the moment of the forces S_s on one side of the section about the center of moments will be zero (since there will be an even number whose resultant is zero), and they need not be considered in the equation of moments. In the present case, the planes

may be passed between a and b , d and f , h and h' , etc. Thus, for the stress in ab , the truss may be cut by a vertical plane in panel ab , or in panel de , and the center of moments taken at B or C . With the center of moments at B , the stress in ab is

$$\frac{R_1' \times p}{\frac{h}{2}} = \frac{R_1 \times 2p}{h}$$

and, with the center of moments at C and section p , the stress in ab is, as before,

$$\frac{R_1 \times 2p - \frac{W}{2} \times p + \frac{W}{2} \times p}{h} = \frac{R_1 \times 2p}{h}$$

In this case, the load at D , being on the right of the center of moments, has a positive, or right-handed, moment, and similar cases must be carefully treated in order to get the signs correct. The stress in any chord member may be found in a manner similar to that just given.

EXAMPLE FOR PRACTICE

If the sixteen-panel through double Warren truss shown in Fig. 18 (*a*) has a span length of 192 feet and a height of 30 feet, and the dead load is 1,200 pounds per linear foot of bridge, all applied at the joints of the loaded chord, what are the dead-load stresses in the members CE , ef , GI , Ff , CD , eF , and HI ?

Ans	{	MEMBER	STRESS, IN POUNDS
		CE	+ 57,600
		ef	- 77,800
		GI	+ 92,200
		Ff	- 7,200
		CD	- 27,600
		eF	+ 13,800
		HI	0

THE MULTIPLE-SYSTEM WARREN OR LATTICE TRUSS

36. Description.—When it is desirable to build a very deep truss of the Warren type, an economical inclination of diagonal and panel length may be used by adding two or three additional systems of web members to the simple type of truss, and subdividing the main panels into three or four

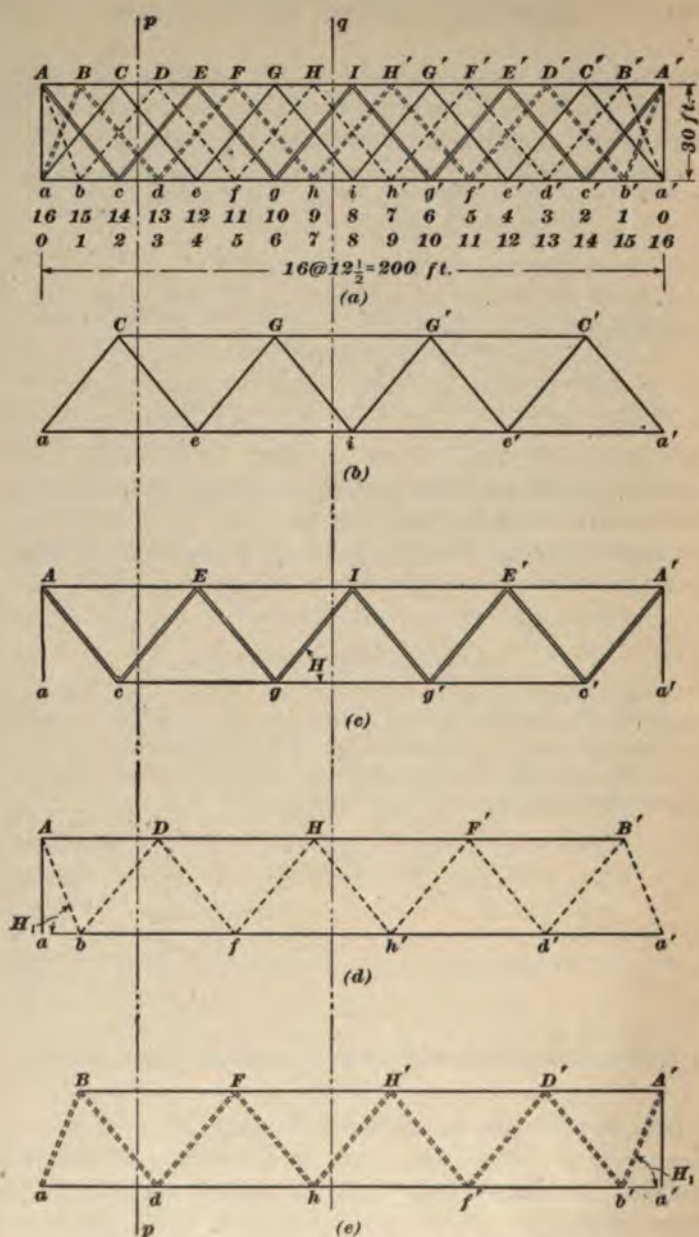


FIG. 19

equal panels. When there are four systems, as shown in Fig. 19 (a), the truss is called a **quadruple-system**, or **quadruple-intersection, Warren truss**. All trusses of this type are included under the general heading of "multiple-system" or "lattice trusses," and are usually built as riveted trusses. The web members of the different systems are riveted together at their intersections. Fig. 19 (a) represents a quadruple-intersection Warren truss with the four systems shown in single and double full and dotted lines. For convenience of reference, the system shown in single full lines in Fig. 19 (b) will be called the *primary system*; that shown in double full lines, Fig. 19 (c), the *secondary system*; that shown in single dotted lines, Fig. 19 (d), the *tertiary system*; and that shown in double dotted lines, Fig. 19 (e), the *quaternary system*.

37. Analysis of Stresses.—It is customary to assume that the only stresses in each system are those caused by the loads directly applied to it. Although this assumption is not strictly correct, it is probably as close as any assumption that can be made concerning the distribution of the stresses. The effect of connecting the web members to each other at every intersection is ignored in the calculation of stresses.

The stresses may be found in the same general way as the stresses in the double Warren truss, by dividing the truss into separate systems and calculating the stresses in the members of each system due to the loads that come on it. The stresses in those members (such as web members) that occur in only one system are the actual stresses; in those that occur in more than one system, they are component stresses, and the actual stresses are found by properly combining the component stresses in the different systems in which the members occur. The analytic method is best adapted to the determination of the stresses.

38. Stresses in Primary System.—Fig. 19 (b) shows the primary system, which is a single-intersection four-panel symmetrical Warren truss supported at the points *a* and *a'*.

The maximum and minimum stresses are found in the ordinary way; the stresses in the web members are the actual stresses in these members. The stresses in the chord members are component stresses.

39. Stresses in Secondary System.—Fig. 19 (*c*) shows the secondary system, which is a single-system symmetrical Warren truss supported at the points A and A' by the verticals aA and $a'A'$. The maximum and minimum stresses are found in the ordinary way; the stresses in the members aA and $a'A'$, and the stresses in the chord members are component stresses, as these members occur in more than one system; the stresses in the inclined web members are the actual stresses.

40. Stresses in Tertiary System.—Fig. 19 (*d*) shows the tertiary system, which is an unsymmetrical single-system Warren truss in which the end diagonals slope differently from the others. The truss is supported at the point A by the vertical aA , and at the point a' by the abutment. The maximum and minimum stresses are found in the same way as for the single-system Warren truss. The stress in aA is equal to the left reaction. The stress in any chord member is equal to the bending moment due to the loads on the system, at the joint opposite the member, divided by the height of the truss. The stress in any web member is equal to the shear in the panel in which the member is located, multiplied by the cosecant of the angle that the member makes with the horizontal. As this system is unsymmetrical, it is necessary to find the dead-load stress and the maximum and minimum live-load stresses in every member, instead of finding them for only those members that are situated on one side of the center, as heretofore.

The maximum live-load chord stresses will occur when the truss is fully loaded; the maximum live-load stress in any web member due to positive shear, when all joints to the right of the member are loaded; the maximum live-load web stresses due to negative shear, when all joints to the left of the member are loaded.

41. Stresses in Quaternary System.—Fig. 19 (*e*) shows the quaternary system, which is the same as the tertiary system turned end for end. The remarks made in connection with the tertiary system apply here. The maximum and minimum stresses in any member of the quaternary system are equal to the stresses in the corresponding member at the other end of the tertiary system. Thus, the stress in $D'b'$ is equal to the stress in Db .

42. Stresses in End Members.—In calculating the actual stress in the end diagonals, such as aB , Ab , etc., it should be remembered that the angle H_1 , Fig. 19 (*d*) and (*e*), for these members is different from the angle H for the remaining diagonals, and that, therefore, the cosecant and cotangent will have different values for the two angles. If the stresses are found by the method of joints, it should be noted that $\cot H_1 = \frac{\cot H}{2}$, and that, then,

$$\begin{aligned} \text{hor. comp. in } aB &= \text{vert. comp. in } aB \times \frac{\cot H}{2} \\ &= \frac{\text{vert. comp. in } aB}{2} \times \cot H \end{aligned}$$

This gives the value of the horizontal component in the end diagonal to be used in finding chord stresses by the method of joints, as explained in Art. 18.

43. Actual Stresses.—The stresses found for the diagonals in the different systems are the actual stresses in the diagonals. The member aA occurs in two systems, and the actual stress in it is equal to the sum of the stresses in aA , as found in the two systems. The chord member AB occurs in two systems; BC , in three; CD , in four; etc.

44. Determination of Stresses by Method of Sections.—To illustrate the calculation of the stresses by the method of sections, let Fig. 19 (*a*) be a sixteen-panel through bridge having a span length of 200 feet and a height of 30 feet; let the dead load be 1,200 pounds per linear foot, one-third of which is assumed to be applied at the joints of the unloaded chord. Let it be required to calculate the

dead-load stresses in all the members cut by a plane at the section $p-p$. This plane is extended so that it will cut all the separate systems represented in Fig. 19 (b), (c), (d), and (e), and to each system the general method of sections is applied independently. When there is a large number of joints, as in this case, it is convenient to number them from each end of the truss, as shown in the lower part of Fig. 19 (a).

The dead panel load is equal to

$$\frac{1,200 \times 12.5}{2} = 7,500 \text{ pounds}$$

of which there is 5,000 at each of the joints b, c, d , etc., and 2,500 at each of the joints B, C, D , etc. The half top-panel loads at A and A' will only affect the stresses in aA , and $a'A'$; they need not be considered in this example. The figure gives

$$\csc H = \frac{\sqrt{30^2 + 25^2}}{30} = \frac{39.05}{30} = 1.30$$

In the primary system, Fig. 19 (b), the left reaction is $\frac{5,000(4 + 8 + 12) + 2,500(2 + 6 + 10 + 14)}{16} = 12,500$ pounds,

and the stresses in the various members are as follows:

MEMBER	STRESS, IN POUNDS
Ce	$(12,500 - 2,500) \times 1.30 = -13,000$
CG	$\frac{12,500 \times 50 - 2,500 \times 25}{30} = +18,800$
ae	$\frac{12,500 \times 25}{30} = -10,400$

In the secondary system, Fig. 19 (c), the left reaction is $\frac{5,000(2 + 6 + 10 + 14) + 2,500(4 + 8 + 12)}{16} = 13,750$ pounds,

and the stresses in the various members are as follows:

MEMBER	STRESS, IN POUNDS
cE	$(13,750 - 5,000) \times 1.30 = +11,400$
AE	$\frac{13,750 \times 25}{30} = +11,500$
cg	$\frac{13,750 \times 50 - 5,000 \times 25}{30} = -18,800$

In the tertiary system, Fig. 19 (*d*), the left reaction is

$$\frac{5,000(3 + 7 + 11 + 15) + 2,500(1 + 5 + 9 + 13)}{16}$$

$$= 15,625 \text{ pounds,}$$

and the stresses in the various members are:

MEMBER	STRESS, IN POUNDS
<i>b D</i>	$(15,625 - 5,000) \times 1.30 = + 13,800$
<i>A D</i>	$\frac{15,625 \times 12.5}{30} = + 6,500$
<i>b f</i>	$\frac{15,625 \times 37.5 - 5,000 \times 25}{30} = - 15,400$

In the quaternary system, Fig. 19 (*e*), the left reaction is

$$\frac{5,000(1 + 5 + 9 + 13) + 2,500(3 + 7 + 11 + 15)}{16}$$

$$= 14,375 \text{ pounds,}$$

and the stresses are:

MEMBER	STRESS, IN POUNDS
<i>B d</i>	$(14,375 - 2,500) \times 1.30 = - 15,400$
<i>B F</i>	$\frac{14,375 \times 37.5 - 2,500 \times 25}{30} = + 15,900$
<i>a d</i>	$\frac{14,375 \times 12.5}{30} = - 6,000$

The actual stresses in the various members are as follows:

MEMBER	STRESS, IN POUNDS
<i>b D</i>	+ 13,800
<i>c E</i>	+ 11,400
<i>C e</i>	- 13,000
<i>B d</i>	- 15,400
<i>C D</i>	sum of stresses in <i>A D</i> , <i>A E</i> , <i>B F</i> , <i>C G</i> , = $6,500 + 11,500 + 15,900 + 18,800 = + 52,700$
<i>c d</i>	sum of stresses in <i>a d</i> , <i>a e</i> , <i>b f</i> , <i>c g</i> , = $- 6,000 - 10,400 - 15,400 - 18,800 = - 50,600$

EXAMPLES FOR PRACTICE

1. Using the same dead loads and dimensions as in Fig. 19, and a live load of 2,000 pounds per linear foot, determine the maximum and minimum stresses due to combined live and dead load in all the members cut by a plane at the section q .

Ans.	MEMBER	STRESS, IN POUNDS	
		MAXIMUM	MINIMUM
	HI	+ 268,800	+ 99,500
	Hh'	- 11,000	+ 5,300
	gI	+ 9,750	- 6,500
	Gi	- 15,400	+ 800
	$H'h$	- 11,000	+ 5,300
	hi	- 256,300	- 97,400

2. With the same data as in example 1, find the maximum stress in aA due to combined live and dead load. Assume that the half-dead panel load at A is included in the secondary system.

Ans. + 85,000 lb.

3. With the same data as in example 1, find, by the method of joints, the dead-load stress in the member FG .

Ans. + 90,100 lb.

THE PRATT TRUSS

45. Description.—The Pratt truss, Fig. 20 (a), is a simple type of truss in which the web members are alternately vertical and inclined; there is a vertical web member at each panel point and an inclined member in each panel, connecting the top of one vertical with the bottom of the next. The Pratt truss is used in deck, through, and half-through bridges; is built pin-connected more frequently than riveted; and is especially adapted to span lengths of 100 to 250 feet. For the longer spans, multiple systems of web or subdivided panels are sometimes built, although such forms are going out of use.

46. Diagonals.—In all except the end panels, *the diagonals are designed to resist tension only*. It was shown in the analysis of the Warren truss that positive shear causes tension in diagonals that slope upwards to the left, and negative shear tension in diagonals that slope upwards to the right. Therefore, in order to have the diagonals in tension, they must slope upwards to the left in the panels where the

shear is positive, and upwards to the right in the panels where the shear is negative. In the panels near the center, in which the maximum combined shear is opposite to the minimum shear, two diagonals will be required, one sloping in each direction. In a panel where there are two diagonals, one of them, the main diagonal, will be in tension when the combined shear is a maximum; the other, called a **counter**, will be in tension when the combined shear is a minimum. When any loading causes tension in one of these diagonals,

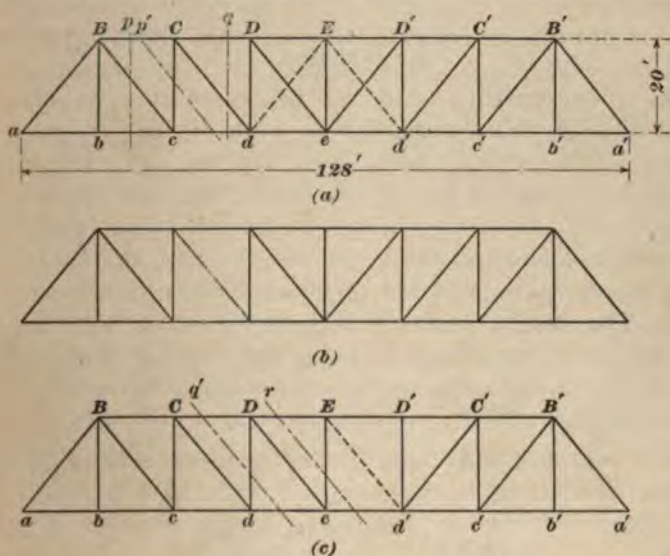


FIG. 20

that diagonal is said to be in action, and the stress in the other is assumed to be zero; the latter diagonal is said to be out of action and need not be considered as part of the truss for that loading. For example, when there is a full load on the truss shown in Fig. 20 (a), the shear in panel de is positive; there is tension in the main diagonal De , and the counter dE is out of action; the shear in panel ed' is negative, there is tension in the main diagonal ed' , and the counter Ed' is out of action. The members of the truss that are in action for full load are shown in Fig. 20 (b); as

the counters are out of action, they are omitted from the diagram.

47. Method of Calculation.—The stresses in the members of the simple Pratt truss can be found analytically or graphically. The work of calculation by either of the analytic methods is so simple that the graphic method will not be employed. The work of calculation can be best illustrated by the consideration of special cases.

THE THROUGH PRATT TRUSS WITH AN EVEN NUMBER OF PANELS

48. Description.—Fig. 20 (*a*) represents an eight-panel through Pratt truss with an even number of panels, the vertical member Ee being the center vertical. The span is 128 feet, and the height 20 feet. The end posts aB and $a'B'$ are compression members; the verticals Bb and $B'b'$ are tension members, and are called the **hip verticals** or **end suspenders**; all other verticals are compression members. The members dE and Ed' are the counters.

It will be assumed that the dead load is 800 pounds, all of which is applied at the joints of the loaded chord, and that the live load is 2,000 pounds per linear foot of bridge.

49. Panel Loads and Reactions.—The dead panel load is

$$\frac{800}{2} \times 16 = 6,400 \text{ pounds,}$$

and each dead-load reaction is

$$\frac{6,400 \times 7}{2} = 22,400 \text{ pounds}$$

The live panel load is

$$\frac{2,000}{2} \times 16 = 16,000 \text{ pounds,}$$

and each live-load reaction for full load is

$$\frac{16,000 \times 7}{2} = 56,000 \text{ pounds}$$

50. Chord Stresses.—To calculate the stress in any chord member, the truss may be considered cut by a surface

That intersects three members, one of which is the member whose stress is desired. The stress in the member is then equal to the bending moment on the truss at the intersection of the two other members, divided by the height of the truss. For example, for the member BC , Fig. 20 (*a*), the truss may be cut by the plane p , or by the plane p' ; in either case, the center of moments is at c , and the stress in BC is equal to the bending moment at c divided by the height.

For the member cd , the truss may be cut by either p' or q ; in either case, the center of moments is at C . As C is vertically over c , the bending moments at these two points are the same; therefore, the stress in BC is numerically equal to the stress in cd . This may also be proved by applying the equation $\sum X = \sum S \cos H = 0$ to all the forces to the left of section p' : as the only horizontal forces are the stresses in BC and cd , they must be equal and opposite. In like manner, it may be shown that the stress in CD is equal to the stress in de .

In calculating the moments at the various points, the work may be simplified by taking the panel length as the unit of length, that is, by expressing the lever arms in panel lengths, and multiplying the result by the panel length in feet. Thus, the moment of R_1 about d may be written $R_1 \times 3$, the distance from the line of action of R_1 to d being 3 panels. Likewise, the moments of the loads at b and c , with respect to d , are, respectively, $W \times 2$ and $W \times 1$. The resultant moment, referred to the panel length as the unit of length, is

$$R_1 \times 3 - W \times 2 - W \times 1$$

The moment (in foot-pounds or foot-tons, as the case may be) is obtained by multiplying this result by the panel length, 16; thus,

$$\text{moment at } d = (R_1 \times 3 - W \times 2 - W \times 1) \times 16$$

The dead-load chord stresses are as follows:

Stress in ab, bc (center of moments at B),

$$\frac{(22,400 \times 1) \times 16}{20} = -17,900 \text{ pounds}$$

Stress in cd (center of moments at C),

$$\frac{(22,400 \times 2 - 6,400 \times 1) \times 16}{20} = -30,700 \text{ pounds}$$

Stress in de (center of moments at D),

$$\frac{[22,400 \times 3 - 6,400(2 + 1)] \times 16}{20} = -38,400 \text{ pounds}$$

Stress in BC (stress in cd), $+30,700$ pounds.

Stress in CD (stress in de), $+38,400$ pounds.

Stress in DE (center of moments at e),

$$\frac{[22,400 \times 4 - 6,400(3 + 2 + 1)] \times 16}{20} = +41,000 \text{ pounds}$$

The dead-load chord stresses are shown in Fig. 21.

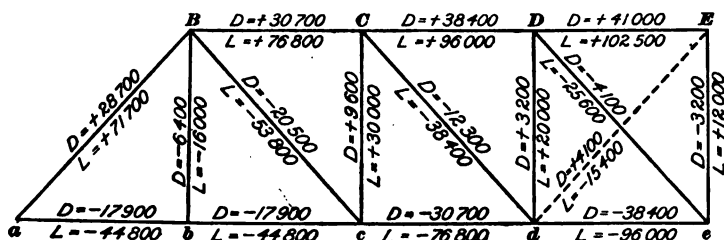


FIG. 21

The live-load chord stresses may be found as above; but, as the dead load is all applied at the joints of the loaded chord, they are more conveniently determined by multiplying the dead-load stresses by the ratio of live to dead load, which is $\frac{2,000}{800}$, or $\frac{5}{2}$. The results are as follows:

MEMBER	STRESS, IN POUNDS
ab, bc	$17,900 \times \frac{5}{2} = -44,800$
cd	$30,700 \times \frac{5}{2} = -76,800$
de	$38,400 \times \frac{5}{2} = -96,000$
BC	$30,700 \times \frac{5}{2} = +76,800$
CD	$38,400 \times \frac{5}{2} = +96,000$
DE	$41,000 \times \frac{5}{2} = +102,500$

The live-load chord stresses are shown in Fig. 21, and the combined chord stresses are shown in Fig. 22. The

maximum stresses are obtained by adding the dead- and live-load stresses; the minimum stresses are the dead-load stresses.

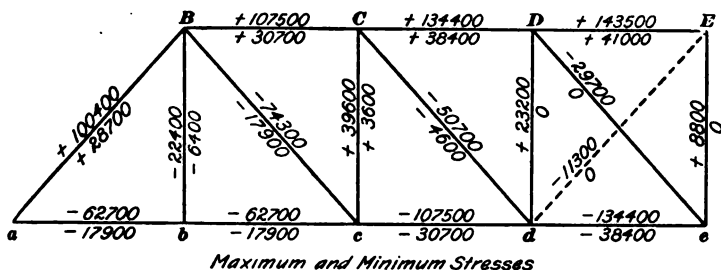


FIG. 22

51. Dead-Load Shears.—As the chords are horizontal, the vertical component of the stress in any web member is equal to the shear on the plane of section that cuts such member and two chord members. Thus, the vertical component in Cd , Fig. 20 (*a*), is equal to the shear on section q . Since in this case there is no load at C , the shear on p' is equal to the shear on q , and it follows that the stress in Cc is equal to the vertical component of the stress in $C'd$. Likewise, the stress in Dd is equal to the vertical component in $D'e$.

The dead-load shears are as follows:

PANEL	SHEAR, IN POUNDS
ab	+ 22,400
bc	+ 16,000
cd	+ 9,600
de	+ 3,200
ed'	— 3,200

and likewise for the remaining panels.

52. Live-Load Shears.—The approximate maximum positive live-load shear in any panel occurs when the truss is loaded on the *right* of the panel (see *Stresses in Bridge Trusses*, Part 1). The approximate values of the maximum positive live-load shears are as follows:

Panel ab (loads at b, c, d, e, d', c' , and b'),

$$\frac{16,000 (1 + 2 + 3 + 4 + 5 + 6 + 7)}{8} = 56,000 \text{ pounds}$$

Panel bc (loads at c, d, e, d', c' , and b'),

$$\frac{16,000 (1 + 2 + 3 + 4 + 5 + 6)}{8} = 42,000 \text{ pounds}$$

Panel cd (loads at d, e, d', c' , and b'),

$$\frac{16,000 (1 + 2 + 3 + 4 + 5)}{8} = 30,000 \text{ pounds}$$

Panel de (loads at e, d', c' , and b'),

$$\frac{16,000 (1 + 2 + 3 + 4)}{8} = 20,000 \text{ pounds}$$

Panel ed' (loads at d', c' , and b'),

$$\frac{16,000 (1 + 2 + 3)}{8} = 12,000 \text{ pounds}$$

Panel $d'c'$ (loads at c' and b'),

$$\frac{16,000 (1 + 2)}{8} = 6,000 \text{ pounds}$$

Panel $c'b'$ (load at b'),

$$\frac{16,000 \times 1}{8} = 2,000 \text{ pounds}$$

In $b'a'$, there can be no positive shear.

The maximum negative shear in any panel is numerically the same as the maximum positive shear in the corresponding panel at the other end of the truss.

53. Combined Shears.—By combining dead-load with positive and negative live-load shears for the left half of the truss, the following results are obtained:

Panel	Dead-Load Shear Pounds	Positive Live-Load Shear Pounds	Negative Live-Load Shear Pounds	Maximum Shear (Dead + Positive Live-Load) Pounds	Minimum Shear (Dead + Negative Live-Load) Pounds
ab	+ 22,400	+ 56,000		+ 78,400	+ 22,400
bc	+ 16,000	+ 42,000	— 2,000	+ 58,000	+ 14,000
cd	+ 9,600	+ 30,000	— 6,000	+ 39,600	+ 3,600
de	+ 3,200	+ 20,000	— 12,000	+ 23,200	— 8,800

54. Exact Live-Load Shears.—For purposes of comparison, the exact maximum live-load shears may be

calculated from the formula given in *Stresses in Bridge Trusses*, Part 1; namely,

$$V'' = W'' \times \frac{m^2}{2(n-1)}$$

In the present case, W'' , the panel load, is equal to 16,000 pounds, and n is equal to 8. For the panel ab , $m = 7$; for the panel bc , $m = 6$; etc. The exact shears, the approximate shears, and the differences are as follows:

Panel	Exact Shear Pounds	Approximate Shear Pounds	Difference Pounds
ab	56,000	56,000	
bc	41,100	42,000	900
cd	28,600	30,000	1,400
de	18,300	20,000	1,700
ed'	10,300	12,000	1,700
$d'c'$	4,600	6,000	1,400
$c'b'$	1,100	2,000	900
$b'a'$			

The approximate shears are greater than the exact in every panel except the two end panels. This is on the safe side, and, as the differences are not great, the approximate values may be used.

Art 55. Maximum and Minimum Stresses in the Diagonals.—The maximum stresses in the diagonals are found from the maximum combined shears. The stress in any diagonal is equal to the combined shear in the panel in which the member is located multiplied by $\csc H$, which in this case is

$$\frac{\sqrt{16^2 + 20^2}}{20} = 1.28$$

In all panels except de , both the maximum and minimum combined shears are positive. In panel de , they are of opposite kinds; when the truss is loaded on the right of this panel, the combined shear is positive; when loaded on

the left, it is negative. This reversal of shear requires the use of two diagonals in panel de . As explained in Art. 46, the diagonal De , which slopes upwards toward the left, is the main diagonal and is in tension when the shear in panel de is positive; the diagonal dE , which slopes upwards toward the right, is the counter, and is in tension when the shear is negative. The maximum combined shear in the panel de is positive and equal to 23,200 pounds; then, the maximum tension in De is equal to $23,200 \times 1.28 = 29,700$ pounds. The minimum combined shear in panel de is negative and equal to 8,800 pounds; then the maximum tension in dE is equal to $8,800 \times 1.28 = 11,260$ pounds. The minimum stress in each diagonal in panel de is equal to zero, as it is assumed that when one is in action the other is out of action.

56. From the foregoing, it follows that the maximum tension in any main diagonal to the left of the center is equal to the maximum positive combined shear in the panel in which the diagonal is located, multiplied by $\csc H$. When the minimum combined shear in any panel is positive, the minimum tension in the diagonal in that panel is equal to the minimum combined shear multiplied by $\csc H$; when the minimum combined shear is negative, a counter is required, the maximum tension in which is equal to the minimum combined shear multiplied by $\csc H$; in the latter case, the minimum tension in both the main diagonal and the counter are equal to zero.

57. Maximum Stresses in the Verticals.—The stress in the hip vertical Bb is found by considering the joint b ; the maximum combined stress in Bb is tension, and equal to the sum of a dead and a live panel load, or 22,400 pounds. The stress in Cc is found by considering joint C . As the only vertical forces acting at this joint are the stresses in Cc and Cd , the maximum combined stress in Cc is compression and equal to the vertical component of the maximum combined stress in Cd , or 39,600 pounds. In like manner, the maximum combined stress in Dd is found to be 23,200 pounds compression, and in Ee , 8,800 pounds compression.

The stress in Ee is a maximum when joints d' , c' , and b' are loaded with live load. Under this condition of loading, the combined shear in panel ed' is positive, counter Ed' is in action, and main diagonal eD' is out of action. The members of the truss that are in action for this loading are shown in Fig. 20 (*c*). The stress in Ee is equal to the shear on section r , which is the maximum positive shear in panel ed' , or $12,000 - 3,200 = + 8,800$ pounds compression.

When a portion of the dead load is assumed to be applied at the joints of the unloaded chord, the stress in any vertical to the left of the center, except the hip vertical, is equal to the vertical component of the stress in the diagonal in the panel to the right, plus the load at the joint of the unloaded chord.

58. Minimum Stresses in the Verticals.—The minimum stress in Bb is tension, and equal to a dead panel load, or 6,400 pounds. The combined stress in Cc is compression, and equal to the vertical component of the minimum combined stress in Cd , or 3,600 pounds. The minimum stress in Dd is zero, and occurs when the stress in the diagonal De is a minimum, that is, when the counter dE is in action and the main diagonal De is out of action. In like manner, the minimum stress in Ee is zero, and occurs when the main diagonals De and eD' are both in action, the stresses in the counters dE and $d'E$ being zero.

59. Combined Stresses.—The maximum and minimum stresses are shown in Fig. 22; the dead and live-load stresses, in Fig. 21. In the counter dE , Fig. 21, there is shown a compressive stress of 4,100 pounds due to dead load; and in Ee , a tensile stress of 3,200 pounds. These are the stresses that would occur in dE and Ee if the main diagonal De in the panel de were omitted when there is no live load on the truss. No compression can actually occur in dE , and no tension in Ee , the values given being the amounts by which the live-load stresses in those members are reduced by the dead load when the main diagonal De is out of action.

60. Odd Number of Panels.—The Pratt truss represented in Fig. 23 has an odd number of panels. In this truss, there is a center panel in which the dead-load shear is zero.

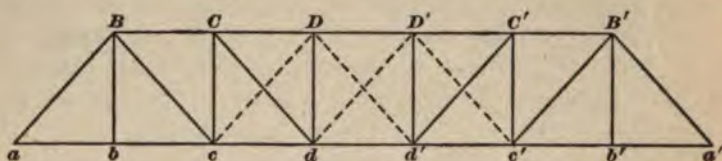


FIG. 23

The dead-load stresses in the two diagonals in this panel are, therefore, equal to zero, and both may be considered as counters. Otherwise, this truss is the same as the one already analyzed.

61. Chord Stresses for Partial Load.—It is sometimes necessary to compute the stress in a chord member due to a live load over a portion of the span. In the panels where there are no counters, the center of moments for any chord member will be the same for a partial as for a full load. In the panels where there are counters, the center of moments for the chord members depends on whether the counter or the main diagonal is in action for the specified loading. If the main diagonal is in action, the center of moments will be at the intersection of the main diagonal with the opposite chord; if the counter is in action, the center of moments will be at the intersection of the counter with the opposite chord.

EXAMPLE.—Let it be required to calculate the stress in the chord member ed' , if the through Pratt truss shown in Fig. 20 (*a*) carries a dead load of 6,400 pounds at each panel point, and a live load of 16,000 pounds at each of the panel points e , d' , c' , and b' .

SOLUTION.—The dead-load reaction at the left end is

$$\frac{7 \times 6,400}{2} = 22,400 \text{ lb.}$$

The dead-load shear in the panel ed' is

$$22,400 - 4 \times 6,400 = -3,200 \text{ lb.}$$

The live-load reaction at the left end is

$$\frac{16,000 \times (1 + 2 + 3 + 4)}{8} = 20,000 \text{ lb.}$$

The live-load shear in the panel ed' is

$$20,000 - 16,000 = +4,000 \text{ lb.}$$

The combined shear in the panel ed' is

$$-3,200 + 4,000 = +800 \text{ lb.}$$

The combined shear is positive, and causes tension in the member sloping upwards to the left, which is the counter $d'E$. Therefore, the counter $d'E$ is in action for the specified loading, as shown in Fig. 20 (c), and the center of moments for ed' is at E , the intersection of $d'E$ and ED' . Then, the stress in ed' is equal to the moment at E divided by the height of the truss; or, the tension in ed' is

$$\frac{[22,400 \times 4 - 6,400(1 + 2 + 3) + 20,000 \times 4] \times 16}{20} = 104,960 \text{ lb. Ans.}$$

st. Term
is here

THE DECK PRATT TRUSS

62. Description.—The deck Pratt truss may be supported in any of the ways shown in Figs. 24, 25, and 26.

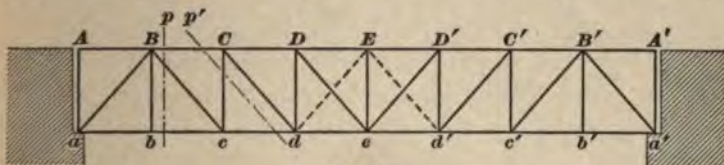


FIG. 24

These three trusses are alike except for the end panels. In Fig. 24, the truss is supported at the point a in the same way

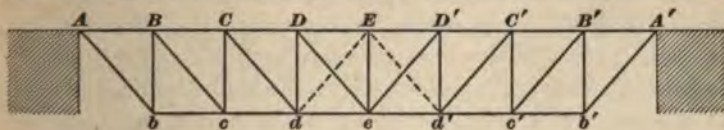


FIG. 25

as the through Pratt truss; the loads in the end panel are supported by stringers AB , the ends of which either rest on

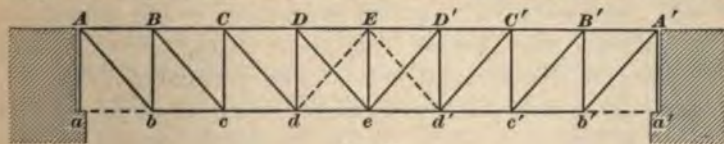


FIG. 26

the masonry at A or are supported by vertical members aA . In this truss the inclined member aB is the end post, and is

a compression member. In Fig. 25, the truss is supported at the point A , and the end diagonal Ab is a tension member. In Fig. 26, the truss is supported at the point a ; the vertical aA is in compression, and is called the *vertical end post*; the web diagonal Ab is in tension; the stress in ab due to dead and live loads is zero, this member being inserted for the sake of stiffness.

63. Chord Stresses.—Using the same loads and dimensions as for the through Pratt truss in Fig. 20 (a), the stress in any chord member of the deck Pratt truss will be the same as the corresponding member of the through Pratt truss. For example, the stress in CD , Fig. 20 (a), is equal to the moment at d divided by the height of the truss; also, the stress in CD , Figs. 24, 25, and 26, is equal to the moment at d divided by the height of the truss. In Figs. 25 and 26, the stress in AB equals the stress in bc . For any other dimensions and loading, the chord stresses may be found in exactly the same way as for the through Pratt truss. The maximum chord stresses occur when there is a full live load; the minimum, when there is no live load.

64. Stresses in Diagonal Members.—The end diagonal in Fig. 24 is a compression member; the end diagonals in Figs. 25 and 26 are tension members. The vertical component of the maximum stress in the end diagonal is equal to the maximum positive shear in the end panel; the vertical component of the minimum stress is equal to the dead-load shear. The stresses in all the other diagonals may be found in exactly the same way as for the through truss.

65. Stresses in Vertical Members.—The stresses in the verticals are different from those in the corresponding members of the through truss. The vertical Bb , Fig. 24, is the hip vertical; the stress in Bb is equal to zero, or to the dead load at b , if any. The maximum stress in Aa , Fig. 26, is equal to the left reaction when there is a full live load; the minimum stress in Aa is equal to the left reaction when there is no live load. In calculating the

stress in Aa , one-half of a panel load must be applied at A , as this is carried to the abutment by Aa . The stress in any other vertical is, in general, equal to the shear on a plane of section cutting that vertical and two chord members. Thus, the stress in Cc , Fig. 24, is equal to the shear in the plane of section p' . Then, the maximum compression in any vertical on the left of the center (except the hip vertical and vertical end post) is equal to the maximum positive shear in a plane cutting that vertical and the two chord members between which the vertical lies. This occurs when the joint at the top of the member and all joints to the right are loaded.

The maximum positive combined shear in the panel to the left of the center vertical may be less than the sum of a dead and a live panel load, in which case the maximum compression in the vertical will be equal to the sum of a dead and a live panel load. In this case, the shear in the panel to the right of the vertical will be negative, and the main diagonals in both center panels will be in action; as the stresses in the two counters that meet at the top of the center vertical will then be equal to zero, the vertical will simply support the load at its upper joint.

When the minimum combined shear in any panel on the left of the center is positive and *greater* than a dead panel load, the minimum stress in the vertical on the right of such panel is equal to the minimum combined shear on the plane cutting such vertical and the two chord members between which it lies. When the minimum combined shear is positive and *less* than a dead panel load, the shear in the panel to the right of the vertical is negative. The diagonal sloping upwards to the left will be in action in the panel on the left; the diagonal sloping upwards to the right, in the panel on the right, and the stress in the diagonal or diagonals that meet the vertical at the top will be zero. Then, the only vertical forces acting at the top of the vertical member will be the dead panel load and the stress in the member; therefore, the minimum stress in such a vertical member will be equal to a dead panel load.

When the minimum shear in the panel to the left of any vertical is negative, the shear in that panel may have any value between the minimum shear, which is negative, and the maximum shear, which is positive. Therefore, under some conditions of loading, the shear in the panel to the left will be positive and less than a dead panel load, and then the stress in the vertical will, as shown, be a minimum and equal to a dead panel load. In like manner, it may be shown that the minimum stress in any vertical between this member and the center will be equal to a dead panel load.

EXAMPLE.—Let it be required to calculate the maximum and minimum stresses in the verticals of the deck Pratt truss shown in Fig. 25, using the same dimensions and loads as for the through truss shown in Fig. 20 (*a*) and described in Art. 48.

SOLUTION.—The maximum and minimum shears are the same as for the through truss, and are given in Art. 53. They are as follows:

Panel	Maximum Shear Pounds	Minimum Shear Pounds
<i>AB</i>	+ 78,400	+ 22,400
<i>BC</i>	+ 58,000	+ 14,000
<i>CD</i>	+ 39,600	+ 3,600
<i>DE</i>	+ 23,200	— 8,800

The maximum stresses (which are the sum of the dead- and live-load stresses) are as follows:

MEMBER	STRESS, IN POUNDS
<i>Bb</i>	+ 22,400 + 56,000 = + 78,400
<i>Cc</i>	+ 16,000 + 42,000 = + 58,000
<i>Dd</i>	+ 9,600 + 30,000 = + 39,600
<i>Ee</i>	+ 3,200 + 20,000 = + 23,200

The maximum stress in *Ee* is equal to the maximum positive shear in panel *DE*, as this is greater than the sum of a full dead and a full live panel load (22,400 lb.).

The minimum stresses are as follows:

MEMBER	STRESS, IN POUNDS
<i>Bb</i>	+ 22,400
<i>Cc</i>	+ 14,000
<i>Dd</i>	+ 6,400
<i>Ee</i>	+ 6,400

The minimum shear in panel CD is positive and equal to 3,600 pounds; it occurs when joints B and C are loaded. Under this condition of loading, the load at D is a dead panel load, or 6,400 pounds. Then, the shear in panel DE for the same loading is $3,600 - 6,400 = -2,800$ pounds; as this is negative, the counter dE is in action and the stress in De is zero. Therefore, the stress in Dd must be equal to the dead panel load at D , which is equal to 6,400 pounds. In like manner, it may be shown that the minimum stress in Ee is 6,400 pounds.

EXAMPLES FOR PRACTICE

1. Let Fig. 27 be a ten-panel through Pratt truss having a span length of 180 feet and a height of 25 feet. If the dead load is 1,200 pounds, one-third of which is applied at the joints of the unloaded chord, and the live load is 2,400 pounds, per linear foot of bridge,

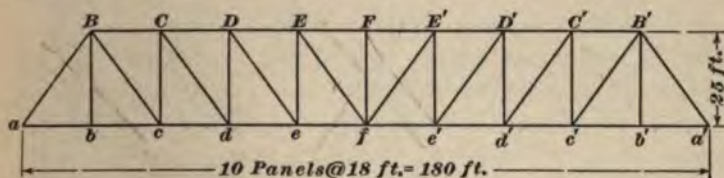


FIG. 27

what are: (a) the maximum stresses in the counters and main diagonals in all the panels in which counters are required? (b) the maximum and minimum stresses due to combined live and dead load in the diagonals aB and Bc ? (c) the maximum and minimum stresses due to combined dead and live load in the verticals Bb , Dd , and Ff ? (d) the maximum and minimum stresses due to combined dead and live load in the chord members bc , BC , and DE ?

	MEMBER	STRESS, IN POUNDS,	
		MAXIMUM	MINIMUM
Ans.	(a) $\begin{cases} Ef, fE' \\ eF, Fe' \end{cases}$	- 46,600	0
	(b) $\begin{cases} aB \\ Bc \end{cases}$	+ 179,600 - 142,400	+ 59,900 - 43,900
	(c) $\begin{cases} Bb \\ Dd \\ Ff \end{cases}$	- 28,800 + 65,160 + 19,800	- 7,200 + 6,840 + 3,600
	(d) $\begin{cases} bc \\ BC \\ DE \end{cases}$	- 105,000 + 186,600 + 279,900	- 35,000 + 62,200 + 93,300

2. Let Fig. 28 be a nine-panel deck Pratt truss having a span length of 180 feet and a height of 26 feet. If the dead load is 1,200 pounds, one-third of which is applied at the joints of the unloaded chord, and the live load is 2,200 pounds, per linear foot of bridge: (a) what are

the maximum combined stresses in the counters and main diagonals in all the panels in which counters are required? (b) what are the

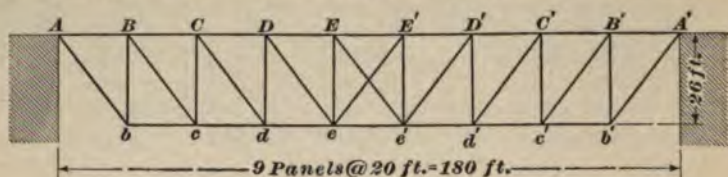


FIG. 28

maximum and minimum combined stresses in the diagonals Ab and Cd ? (c) in the verticals Bb , Dd , and Ee ? (d) in the chord members bc , CD , and EE' ?

	MEMBER	STRESS, IN POUNDS	
		MAXIMUM	MINIMUM
Ans.	(a) $\begin{cases} Ee', eE' \\ Dd, dD' \\ dE, d'E' \end{cases}$	- 30,800	0
		- 61,400	0
		- 3,400	0
	(b) $\begin{cases} Ab \\ Cd \end{cases}$	- 171,600	- 60,600
		- 95,100	- 21,000
	(c) $\begin{cases} Bb \\ Dd \\ Ee \end{cases}$	+ 132,000	+ 44,000
		+ 71,300	+ 12,700
		+ 44,700	+ 8,000
	(d) $\begin{cases} bc \\ CD \\ EE' \end{cases}$	- 104,600	- 36,900
		+ 235,400	+ 83,100
		+ 261,500	+ 92,300

THE HOWE TRUSS

66. Description.—The Howe truss, shown in Fig. 29, was one of the earliest forms of simple bridge trusses in America. As originally constructed, it had short panels with

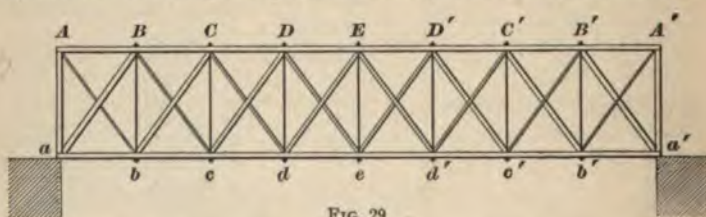


FIG. 29

two diagonals in each panel, and vertical end posts; all parts of the truss were constructed of wood, except the intermediate verticals, which were iron rods. This type of truss is still used to some extent in localities where timber is

plentiful. At present, however, all the members that receive no stress are omitted, and the lower chord is frequently constructed of steel. The diagonals are designed to resist compression only; therefore, they must slope downwards to the left in the panels in which the shear is positive; and downwards to the right in the panels in which the shear is negative. Two diagonals, one sloping in each direction, are required in each panel in which the sign of the maximum combined shear is opposite to that of the minimum combined shear. In the original trusses of this type, two diagonals were put in each panel, but the extra diagonals or counters in the panels near the ends were unnecessary. The verticals, except the vertical end post, were designed to resist tension only.

The modern forms of the Howe truss are shown in Fig. 30 as a through truss, and in Fig. 31 as a deck truss. The end posts, upper chord, and intermediate diagonals are constructed of wood; the verticals and bottom chord are of steel. The counters are shown in dotted lines, and are put in only where necessary.

67. Calculation of Stresses.—The stresses in the members are calculated in exactly the same way as the stresses in the members of the Pratt truss. The maximum stress in the center vertical Dd of the through truss shown in Fig. 30 is equal to the maximum positive shear in panel cd

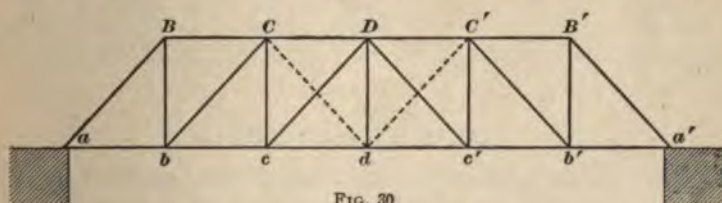


FIG. 30

(when there is no dead load at D), or to a full dead and live panel load, whichever is the greater; the minimum stress is equal to a dead panel load. The maximum stress in the center vertical dD of the deck truss shown in Fig. 31 is equal to the maximum positive shear in the panel dc' ; the

minimum stress is equal to zero, and occurs when the main diagonals Cd and dC' are in action. (In reality, the stress

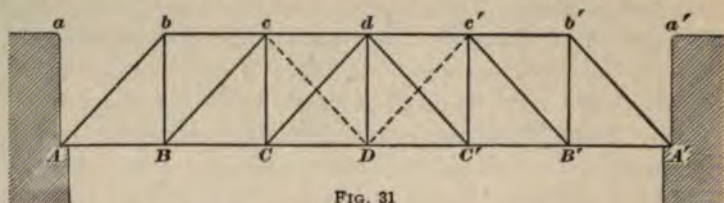


FIG. 31

in dD , under this condition of loading, is equal to the dead load at D ; but, if this is all assumed at the joints of the loaded chord, the stress in dD will be zero.)

EXAMPLE FOR PRACTICE

Let Fig. 30 be a six-panel through Howe truss having a span length of 90 feet, consisting of six 15-foot panels, and a height of 12 feet. If the dead load is 800 pounds, all applied at the loaded chord, and the live load is 2,000 pounds, per linear foot of bridge; find: (a) the maximum and minimum combined stresses in the chord members BC and bc ; (b) the maximum combined stresses in both the main diagonal and the counter in each panel to the left of the center in which a counter is required; (c) the maximum and minimum combined stresses in the verticals Bb and Dd .

Ans.	MEMBER	STRESS, IN POUNDS	
		MAXIMUM	MINIMUM
(a)	BC	+ 65,600	+ 18,800
	bc	- 105,000	- 30,000
(b)	cD	+ 28,800	0
	Cd	+ 7,200	0
(c)	Bb	- 52,500	- 15,000
	Dd	- 21,000	- 6,000

THE WHIPPLE TRUSS

68. Description.—If the simple type of Pratt truss is used for the longer spans to which the Pratt truss is adapted, it will be impossible to make use of an economical height of truss and inclination of diagonal without using very long panels, thereby making the floor system very heavy. For such spans, it is convenient to use a modified form of the Pratt truss, making use of multiple systems of web or

subdivided panels. The **Whipple truss**, sometimes called the **double-intersection Pratt truss**, is a modified form of the Pratt truss, in which there are two systems of web members, the diagonals running across two panels, as shown in Fig. 32 (a). The two systems of web members are shown in full and dotted lines. There is a large number of these trusses in use at the present time, but they are now

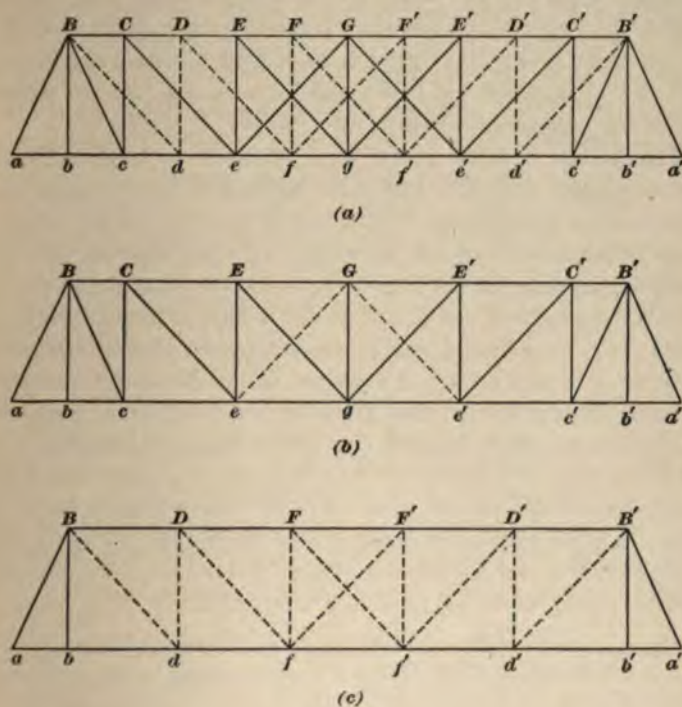


FIG. 32

avoided by the best engineers, because the actual stresses in some members cannot be determined accurately.

69. Method of Calculation.—Applying the principles of the method of sections to the truss shown in Fig. 32 (a), it will be seen that, if the truss is cut by a plane in any panel except the end panel, the plane will cut at least four members in which the stresses are unknown, and it will be

impossible to determine those stresses by the principles of statics, unless some assumption is made regarding the distribution of the stresses among the members. It is customary to assume that the truss is composed of two trusses lying in the same plane and having the chords and end posts in common. The stresses in each truss are supposed to be caused by the loads that are directly applied to it. The two systems in this case are shown separated in Fig. 32 (*b*) and (*c*). For convenience of reference, the system shown in full lines in Fig. 32 (*b*) will be called the *primary system*, and the system shown in dotted lines in Fig. 32 (*c*) will be called the *secondary system*.

The joints *B*, *b*, *B'*, and *b'* are common to both systems, and one-half of each of the loads applied at those points may be assumed to be supported by each system. The stresses may be found in all the members of each system, for the loads that come on that system, in exactly the same way in which the stresses are found in the single-intersection Pratt truss. As the chords, end posts, and hip verticals are common to both systems, the stresses found in those members in each system are partial stresses, or component stresses, and must be added together to get the actual stresses. As each of the web members, except the end posts and hip verticals, occurs in but one system, the stresses found in them are the actual stresses. The assumption that the two web systems act independently is not strictly correct, but the resulting error is probably very small.

EXAMPLE.—Let it be required to calculate the maximum stresses in the chord members *CD*, *DE*, *ef*, and *fg*, and in the web members *Bc*, *Ee*, *eG*, *Dd*, and *Df*, in the twelve-panel through Whipple truss shown in Fig. 32 (*a*), having a span length of 216 feet, and a height of 36 feet, when the dead load is 1,000 pounds (all applied at the joints of the loaded chord), and the live load is 1,600 pounds, per linear foot of bridge. The dead and the live panel load are 9,000 and 14,400 pounds, respectively; at the joints *b* and *b'*, the panel loads for each system are to be taken at one-half these values.

SOLUTION.—*Chord Stresses*.—The truss may be separated into the primary system shown in Fig. 32 (*b*) and the secondary system shown in Fig. 32 (*c*). To find the stresses in *CD*, *DE*, *ef*, and *fg*, it is

necessary to calculate the stresses in CE and eg of the primary system, and in BD , DF , df , and ff' of the secondary system. The total dead load on the primary system is $9,000 \times 6$, and each reaction is equal to

$$\frac{9,000 \times 6}{2} = 27,000 \text{ lb.}$$

The dead-load chord stresses are as follows:

$$\text{Stress in } CE = \frac{(27,000 \times 4 - 4,500 \times 3 - 9,000 \times 2) \times 18}{36} = +38,250 \text{ lb.}$$

The stress in eg is equal to the stress in CE = $-38,250$ lb.

The total dead load on the secondary system is $9,000 \times 5$, and each reaction is equal to

$$\frac{9,000 \times 5}{2} = 22,500 \text{ lb.}$$

The dead-load chord stresses are as follows:

MEMBER	STRESS, IN POUNDS
BD	$\frac{(22,500 \times 3 - 4,500 \times 2) \times 18}{36} = +29,250$
DF	$\frac{(22,500 \times 5 - 4,500 \times 4 - 9,000 \times 2) \times 18}{36} = +38,250$
df (stress in BD)	$-29,250$
ff' (stress in DF)	$-38,250$

Then, the dead-load chord stresses in this truss are:

MEMBER	STRESS, IN POUNDS
CD	$38,250 + 29,250 = +67,500$
DE	$38,250 + 38,250 = +76,500$
ef	$38,250 + 29,250 = -67,500$
fg	$38,250 + 38,250 = -76,500$

As all the dead load is applied at the joints of the loaded chord, the live-load stresses will be equal to the dead-load stresses multiplied by $\frac{1,600}{1,000}$, and the maximum combined stresses will be equal to the dead-load stresses multiplied by $\frac{2,600}{1,000}$, or by 2.6.

The maximum combined stresses are as follows:

MEMBER	MAXIMUM STRESS, IN POUNDS
CD	$+67,500 \times 2.6 = +175,500$
DE	$+76,500 \times 2.6 = +198,900$
ef	$-67,500 \times 2.6 = -175,500$
fg	$-76,500 \times 2.6 = -198,900$

Web Stresses.—The web members Bc , Ee , and eG occur in the primary system. The dead-load shears are as follows:

$$\text{shear in panel } bc = 27,000 - 4,500 = 22,500 \text{ lb.}$$

$$\text{shear in panel } eg = 27,000 - 4,500 - 9,000 \times 2 = 4,500 \text{ lb.}$$

For the member Bc ,

$$\csc H = \frac{\sqrt{18^2 + 36^2}}{36} = 1.118;$$

for the member eG ,

$$\csc H = \frac{\sqrt{36^2 + 36^2}}{36} = 1.414.$$

The dead-load stress in Bc is $22,500 \times 1.118 = -25,200$ lb.

The dead-load stress in Ee is $+4,500$ lb.

The dead-load stress in eG (counter, assuming that the main diagonal Eg is left out) is $4,500 \times 1.414 = +6,400$ lb.

The members Dd and Df occur in the secondary system. The shear in the panel df is $22,500 - 4,500 - 9,000 = 9,000$ lb.

The dead-load stress in Dd is $+9,000$ lb.

The dead-load stress in Df is $9,000 \times 1.414 = -12,700$ lb.

The maximum live-load stresses for the primary system are as follows, the load at b' being a half-panel load: For the member Bc , with the truss loaded to c from the right end, the shear in panel bc is

$$\frac{14,400 (\frac{1}{2} + 2 + 4 + 6 + 8 + 10)}{12} = 36,600 \text{ lb.}$$

The live-load stress in Bc is $36,600 \times 1.118 = -40,900$ lb.

For the member Ee , with the truss loaded to g from the right end, the shear in panel eg is

$$\frac{14,400 (\frac{1}{2} + 2 + 4)}{12} = 15,000 \text{ lb.}$$

The live-load stress in Ee is $+15,000$ lb.

For the counter eG , the member $e'G$ may be considered instead. With the truss loaded to e' from the right end, the shear in panel ge' is

$$\frac{14,400 (\frac{1}{2} + 2 + 4)}{12} = 7,800 \text{ lb.}$$

The live-load stress in $e'G$ is $7,800 \times 1.414 = -11,000$ lb.

The maximum live-load stresses for the secondary system are as follows, the load at b' being a half-panel load: For the members Dd and Df , with the truss loaded to f , from the right end, the shear in panel df is

$$\frac{14,400 (\frac{1}{2} + 3 + 5 + 7)}{12} = 18,600 \text{ lb.}$$

The live-load stress in Dd is $+18,600$ lb.

The live-load stress in Df is $18,600 \times 1.414 = -26,300$ lb.

The final maximum combined stresses are as follows:

MEMBER	STRESS, IN POUNDS
Bc	$-25,200 - 40,900 = -66,100$
Ee	$4,500 + 15,000 = +19,500$
eG	$-11,000 + 6,400 = -4,600$
Dd	$9,000 + 18,600 = +27,600$
Df	$-12,700 - 26,300 = -39,000$

EXAMPLE FOR PRACTICE

Let Fig. 32 (*a*) be a twelve-panel through Whipple truss having a span length of 180 feet and a height of 30 feet. If the dead load is 1,200 pounds, all applied at the joints of the loaded chord, and the live load is 2,200 pounds, per linear foot of bridge, find: (*a*) the maximum and minimum combined stresses in the chord members *cd*, *de*, *EF*, and *FG*; (*b*) the maximum and minimum combined stresses in the web members *Bd*, *Eg*, *Ff'*, *Cc*, and *Ff*.

	MEMBER	STRESS, IN POUNDS	
		MAXIMUM	MINIMUM
Ans.	(<i>a</i>) <i>cd</i>	- 102,000	- 36,000
	<i>de</i>	- 153,000	- 54,000
	<i>EF</i>	+ 229,500	+ 81,000
	<i>FG</i>	+ 229,500	+ 81,000
	(<i>b</i>) <i>Bd</i>	- 73,100	- 24,500
	<i>Eg</i>	- 30,700	0
	<i>Ff'</i>	- 16,500	0
	<i>Cc</i>	+ 41,700	+ 10,100
	<i>Ff</i>	+ 11,700	0

THE POST TRUSS

70. Description.—The Post truss, Fig. 33 (*a*), may be looked on as a modified Whipple truss with an odd number of panels in the bottom chord. This truss is seldom built at present on account of the uncertainty in the stresses. The compression web members are inclined so that their upper joints are one-half a panel nearer the center of the truss than their lower joints. The two center members, one on each side of the center panel of the lower chord, meet at the center of the upper chord. The main diagonals are tension members and slope across one and one-half panels. The diagonals shown in dotted lines are counters, and were formerly inserted in each panel, although those in the panels near the ends were not needed. At present, counters are used only where they are actually needed. There are in reality two systems of web members, but as they are connected at the center joint of the upper chord, and as the counters connect the compression web members of the two systems, it is impossible to calculate the stresses from the equations of equilibrium without making some

assumption as to their distribution. The common assumptions for this truss give only roughly approximate results. It is customary to divide the truss into two systems, as shown in full lines in Fig. 33 (*b*) and (*c*), and treat each system as an independent truss in much the same way as in the Whipple truss. It will be seen that each system is unsymmetrical, but that the two are alike end for end.

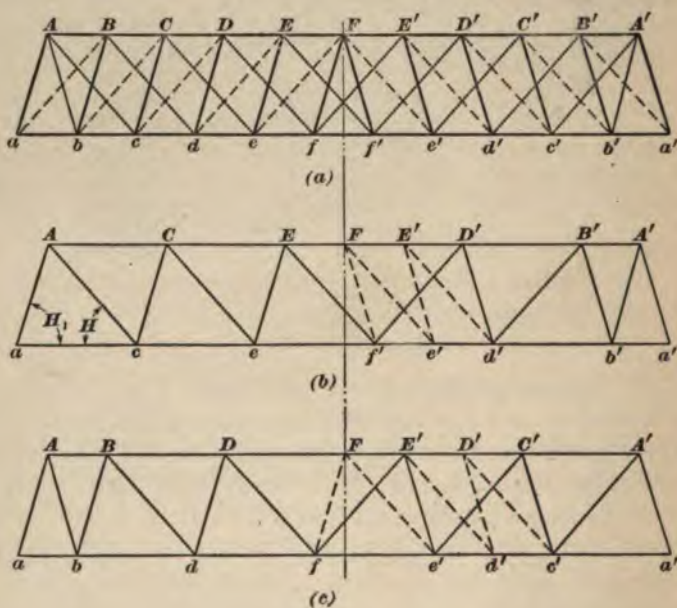


FIG. 33

71. Chord Stresses.—It will be necessary to calculate the stresses in all the chord members of only one system. For example, the maximum stress in AB , Fig. 33 (*a*), is equal to the sum of the stresses in AC , Fig. 33 (*b*), and AB , Fig. 33 (*c*). But, since, when the whole truss is fully loaded, the stress in $A'B'$ is equal to that in AB , Fig. 33 (*c*), it is not necessary to calculate the latter stress separately. The stress in AC is equal to the moment at c divided by the height; the stress in $A'B'$ is equal to the moment at b' divided by the height. In like manner, other chord stresses may be found.

72. Stresses in Main Web Members.—As the systems are alike end for end, it is only necessary to analyze each system for loads on the right of the center. The member Aa , Fig. 33 (*a*), occurs in both systems; the stress in it is equal to the sum of the reactions at a in the two systems, Figs. 33 (*b*) and (*c*), multiplied by $\csc H$; or, since the reaction at a in (*c*) is equal to the reaction at a' in (*b*), the stress in aA is equal to the sum of the two reactions of either system multiplied by $\csc H$. The same is true of the stress in $A'a'$. The maximum stresses in the other web members may be found from the maximum shears in the respective panels. For example, the vertical component of the maximum stresses in Ac and in $A'c'$ is equal to the maximum positive shear in panel ac , Fig. 33 (*b*); in Cc and Ce , $C'c'$ and $C'e'$, to the maximum positive shear in panel ce , Fig. 33 (*b*); in Ee and $E'f$, $E'e'$ and $E'f'$, to the maximum positive shear in panel ef , Fig. 33 (*b*).

Also, the vertical component of the maximum stresses in Ab and $A'b'$ is equal to the maximum positive shear in panel ab , Fig. 33 (*c*); in Bb and Bd , $B'b'$ and $B'd'$, to the maximum positive shear in panel bd , Fig. 33 (*c*); in Dd and Df , $D'd'$ and $D'f'$, to the maximum positive shear in panel df , Fig. 33 (*c*). The maximum positive shear in any panel obtains when all joints to the right are loaded with a live load.

73. Stresses in Counters.—The compression members Ff and Ff' may be considered as counters, as they are not in action for full load. In Fig. 33 (*b*), when the combined shear in panel $f'd'$ is positive, the members $D'd'$ and $D'f'$ may be considered out of action, and the members $d'E'$, $E'e'$, $e'F$, and Ff' in action. In Fig. 33 (*c*), when the combined shear in panel $f'e'$ is positive, the members $e'E'$ and $E'f'$ may be considered out of action, and the members $e'F$ and Ff in action; when the combined shear in panel $e'c'$ is positive, $c'C'$ and $C'e'$ may be considered out of action, and $c'D'$, $D'd'$, $d'E'$, and $E'e'$ in action. When the live load extends to e' from the right, the joints d' and b' ,

Fig. 33 (*b*), and e' and c' , Fig. 33 (*c*), will be loaded. If the combined shear in panel fe' is positive, the vertical component of the stresses in Ff and $F'e'$ is equal to the shear in the panel fe' ; if the combined shear in the panel $f'd'$ is also positive, the vertical component of the stress in Ff' and $E'd'$ is equal to the shear in panel $f'd'$, and the vertical component of the stress in $F'e'$ is equal to the sum of the shears in panels $f'd'$ and fe' . The stresses in the other counters may be found in like manner.

It must be understood that for this truss the foregoing method of calculation is *roughly approximate* and is given here simply to afford a means of finding stresses in trusses of this nature already built rather than as a guide in designing. The Whipple truss answers the same purpose as the Post truss, and is preferable because the approximate method of calculation gives closer results for the former than for the latter.

THE BALTIMORE TRUSS

Here **74. Description.**—The Baltimore truss, shown as a through bridge in Fig. 34, is in reality a Pratt truss with long panels, which are subdivided by the addition of short verticals and diagonals, such as Dd and Dc , intersecting the main diagonals half way between the top and bottom chords. This form of truss allows the use of an economical height and inclination of diagonal, without excessively long panels, and is used to a great extent at the present time for railroad and highway bridges. The short verticals (Dd , Ff , etc.), called the **subverticals**, are tension members; the short diagonals (cD , eF , etc.), called the **substruts**, are compression members; eF also acts as a tension member as the lower half of the counter eFG when that counter is in action. All the other members are similar to the corresponding members of the simple Pratt truss.

75. Chord Stresses.—The stress in any top-chord member may be found by cutting the truss by a plane that intersects the member whose stress is desired and the lower half of a main diagonal. For example, for the stress in CE ,

denoting the panel length by p , and the height of the truss by h .

The stresses in the bottom-chord members ab , de , fg , etc. may be found by cutting the truss by a plane that intersects the member whose stress is desired and the lower half of a main diagonal. For example, for the stress in de , Fig. 34 (a), the truss may be considered as cut by a plane at the section q ; then the stress in de is equal to the moment of all the forces to the left of section q about the point C , divided by the height of the truss. In this case, loads at d and D , if any, have positive or right-handed moments about the point C , similar to the moment of R_1 , and this must be remembered in writing the equation of moments. *The moment of the forces on the left of section q about the point C is not the bending moment on the truss at the point C .* In the present case, Fig. 34 (b),

$$S_2 = \frac{R_1 \times 2p - W \times p + W \times p}{h}$$

The stresses in the bottom-chord members bc , cd , ef , etc. may be most easily found by considering the forces acting at the intermediate panel points. For example, for the stress in cd , Fig. 34 (a), the joint d may be considered a free body, as shown in Fig. 34 (c). The only horizontal forces are S_4 and S_2 ; they must be equal and opposite; in other words, the stress in cd is equal to the stress in de . In like manner, it may be shown that the stress in bc is equal to the stress in ab , that in ef equal to that in fg , etc.

The maximum chord stresses obtain when there is a full live load; the minimum stresses, when there is no live load.

76. Stresses in the Subverticals.—The stress in any subvertical may be found by considering the forces acting at an intermediate panel point. For example, for the stress in Dd , the joint d , Fig. 34 (c), may be considered. The only vertical forces are the stress in Dd and the panel load W ; they must be equal and opposite. Therefore, *the stress in any subvertical is tension; the maximum stress is equal to the sum*

of a dead and a live panel load; the minimum stress is equal to a dead panel load.

77. Stresses in the Short Diagonals or Substruts.

The stresses in the short diagonals Bc , Dc , Fc , etc. may be found by considering the forces acting at the intermediate joints B , D , F , etc. For example, for the stress in cD , Fig. 34 (a), the joint D may be treated as a free body, as shown in Fig. 34 (d). There are four forces (S_2 , S_3 , S_4 , and S_5) acting on joint D ; S_3 is required; S_4 is known; S_2 and S_5 are unknown, and are not required at the present time. S_4 may be resolved at the point c in its line of action into S_y and S_x , its vertical and horizontal components, respectively. If the point c is taken as the center of moments, the moments of S_2 , S_3 , and S_5 will all be zero. Then,

$$\sum M = S_y \times 2p - S_3 \times p = 0;$$

whence

$$S_y = \frac{S_3}{2} = \frac{W}{2}$$

and

$$S_3 = S_y \times \csc H = \frac{W}{2} \csc H, \text{ compression}$$

In other words, the vertical component of the stress in a short diagonal is equal to one-half the panel load at the top and at the foot of the short vertical. Then, *the maximum compression in a short diagonal is equal to one-half the sum of a dead and a live panel load multiplied by $\csc H$; the minimum compression (except in cF) is equal to one-half of a dead panel load multiplied by $\csc H$.* The minimum stress in cF will be discussed later in connection with counter cFG . In case part of the dead load is applied at the point D , this must be added to the panel load at d in getting the stress in cD .

78. Stresses in the Hip Verticals.—The stress in the hip vertical Cc may be found by considering the joint c as a free body [Fig. 34 (e)]. The forces acting at c are W , S_1 , S_2 , S_3 , S_4 , and S_5 . The equation $\sum Y = \sum S \sin H = 0$ gives

$$\sum Y = S_1 - S_2 \sin H - S_3 \sin H - W = 0;$$

whence

$$S_1 = S_2 \sin H + S_3 \sin H + W, \text{ tension}$$

S_s and S_e are the stresses in the short diagonals Bc and cD ; therefore, $S_s \sin H$ and $S_e \sin H$ are each equal to $\frac{W}{2}$, and

$$S_s = \frac{W}{2} + \frac{W}{2} + W = 2W$$

It is thus seen that *the maximum tension in the hip vertical is equal to twice the sum of a dead and a live panel load; and that the minimum tension is equal to twice a dead panel load.*

79. Stresses in the Main Diagonals.—The stress in the lower half of a main diagonal may be found by considering the portion of the structure to the left of a plane cutting that member and two chord members. For example, for the stress in De , Fig. 34 (*a*), the truss may be cut by a plane at the section q . The vertical component of the stress in the diagonal is equal to the shear on the section. The stress in the end post is compression; in the other diagonals, it is tension. The maximum stress in the lower half of a main diagonal to the left of the center is equal to the maximum positive shear multiplied by $\csc H$; the minimum stress is equal to the minimum shear multiplied by $\csc H$, when the minimum shear is positive. When the minimum shear is negative, a counter is required, and the minimum stress in the main diagonal is zero.

The stress in the upper half of a main diagonal may be found by considering the forces acting on the portion of the structure to the left of a plane of section that cuts the upper half of that diagonal, a short diagonal, and two chord members. For example, for the stress in EF , Fig. 34 (*a*), the portion of the truss to the left of section r may be considered [see Fig. 34 (*f*)]. The equation $\Sigma Y = \Sigma S \sin H = 0$ gives

$$\Sigma Y = R_1 - 4W - S_{12} \sin H - S_{11} \sin H = 0;$$

whence

$$S_{11} \sin H = \text{shear on section } r - S_{12} \sin H$$

S_{12} is the stress in the short diagonal eF , and is equal to one-half the panel load at f multiplied by $\csc H$. Then,

$$S_{12} \sin H = \frac{W}{2}$$

and
$$S_{11} = \left(\text{shear on section } r - \frac{W}{2} \right) \times \csc H$$

In other words, *the stress in the upper half of a main diagonal is equal to the algebraic sum of the shear in the panel in which the member is located and the vertical component of the stress in the short diagonal, multiplied by $\csc H$.*

The maximum stress in BC obtains when the truss is fully loaded, and is compression; the vertical component of the stress is equal to the shear in the panel bc plus the vertical component of the stress in Bc . (The load at b increases the vertical component of the stress in BC by $\frac{1}{12}W - W + \frac{6}{12}W = \frac{5}{12}W$.) The minimum stress obtains when there is no live load on the truss.

The maximum stress in CD is tension, and obtains when all the joints from d to b' are loaded with a live load. (The load at d increases the vertical component of the stress in CD by $\frac{9}{12}W - \frac{6}{12}W = \frac{3}{12}W$.) The minimum stress obtains when joints b and c are loaded with live load.

The maximum stress in EF is tension, and obtains when all the joints from f to b' are loaded with live load. (The load at f increases the vertical component of the stress in EF by $\frac{7}{12}W - \frac{6}{12}W = \frac{1}{12}W$.) The minimum stress obtains when all the joints from b to e are loaded with live load.

If the minimum combined shear in panel ef is positive and greater than a dead panel load, the shear in panel fg will also be positive, and the main diagonal gFE will be in action. The stress in EF is then equal to the shear in panel ef minus one-half of the dead load at f , multiplied by $\csc H$. If the minimum shear in the panel ef is negative, or positive and less than the dead load at f , the shear in the panel fg for the same loading will be negative, and the member FG will be in action as a counter. Under this condition, the stress in the lower half of the main diagonal gF will be zero, and the members that are in action at joint F will be those shown in Fig. 34 (*g*). It will be seen that the stress in EF is thus equal to one-half the dead load at F multiplied by $\csc H$; this is the minimum stress in EF , and is tension.

80. Stresses in the Counters.—The stress in eF is a minimum (maximum tension) when the live load extends

from b to e . If the combined shear in the panel ef is then negative, the stress in eF is tension and equal to the shear plus one-half the dead load at f , multiplied by $\csc H$. If the combined shear in panel ef is then positive and less than one-half the dead load at f , the stress in eF is tension and equal to the difference between one-half the dead load at f and the shear in the panel ef , multiplied by $\csc H$. If the shear in panel ef is positive and greater than one-half, but less than a full, panel dead load at f , the stress in eF is compression and equal to the shear in the panel ef minus one-half the dead load at f , multiplied by $\csc H$.

The maximum tension in FG is equal to the maximum negative shear in the panel fg multiplied by $\csc H$, and obtains when the live load extends from b to f . The minimum stress is zero.

81. Stresses in the Verticals.—The maximum stress in Gg is compression, and is equal to the vertical component of the maximum stress in FG plus the dead load at G , if any; the minimum stress is zero, and obtains when the counters FG and $F'G$ are out of action. The maximum stress in Ee is compression, and is equal to the vertical component of the maximum stress in EF ; the minimum stress is equal to the vertical component of the minimum stress in EF .

82. Modified Baltimore Truss.—Fig. 35 shows a modified form of the Baltimore truss, in which the short

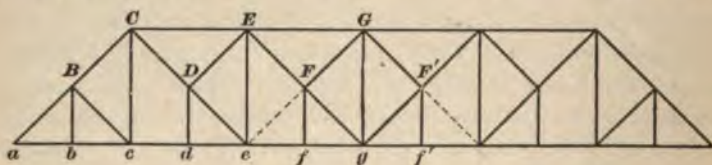


FIG. 35

diagonals (except Bc) are attached to the upper-chord joints, and are tension members. The general method of analysis is the same as for the truss already treated.

83. Deck Truss.—When used in a deck bridge, the Baltimore truss may be supported as shown in Fig. 36.

In this form, the subverticals bB, dD , etc. are compression members, and the short diagonals Bc, De , etc. are tension members. The general method of analysis is the same as for the through truss.

84. Distribution of Dead Load.

If the dead load is assumed to be divided between the loaded and unloaded chords, two-thirds of a dead panel load may be taken at the joints of the loaded chord (b, c, d , etc.), one-third at the joints of the unloaded chord (C, E , etc.), and one-third at the intermediate joints (B, D , etc.), although the latter are not on the unloaded chord. In the preceding discussion, it was assumed for convenience in explanation that all the dead load was applied at the joints of the loaded chord. The actual stresses in the verticals will be slightly different if the dead load is distributed among the several joints; the stresses in the other members will remain the same.

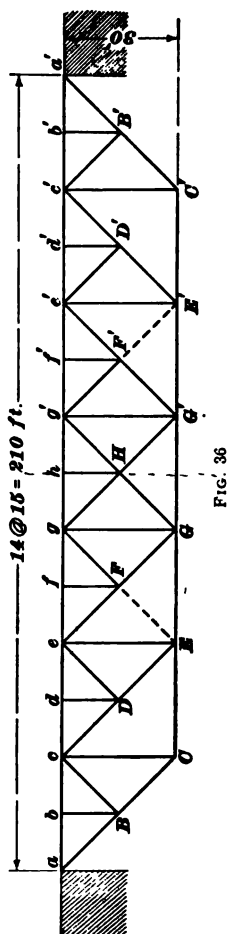
EXAMPLE.—Let Fig. 36 be a fourteen-panel deck Baltimore truss having a span length of 210 feet and a height of 30 feet. If the dead load is 1,200 pounds, two-thirds of which is applied at the loaded chord, and the live load is 2,400 pounds per linear foot, what are the maximum and minimum stresses in all the members?

SOLUTION.—*Panel Loads and Reactions.*
The dead panel load is

$$\frac{1,200}{2} \times 15 = 9,000 \text{ lb.}$$

of which 6,000 lb. is applied at each of the joints b, c, d , etc.; 3,000 lb. at B, D, F , etc.; and 3,000 lb. at C, E, G , etc. The dead-load reaction is equal to

$$\frac{9,000 \times 13}{2} = 58,500 \text{ lb.}$$



The live panel load is equal to

$$\frac{2,400}{2} \times 15 = 18,000 \text{ lb.}$$

and the reactions for a fully loaded truss are each

$$\frac{18,000 \times 13}{2} = 117,000 \text{ lb.}$$

Chord Stresses.—As the loads of 6,000 lb. at *b, c, d*, etc. are vertically over the loads of 3,000 lb. at *B, C, D*, etc., the moment of 6,000 lb. at an upper joint, plus the moment of 3,000 lb. at a lower joint, about any point will be the same as the moment of 9,000 lb. about the same point. The minimum stresses in the chords are the dead-load stresses; they are as follows:

MEMBER	DEAD-LOAD STRESS, IN POUNDS
<i>a b, b c</i>	$\frac{58,500 \times 15}{15} = + 58,500$
<i>c d, d e</i>	$\frac{[58,500 \times 4 - 9,000 (3 + 2)] \times 15}{30} = + 94,500$
<i>e f, f g</i>	$\frac{[58,500 \times 6 - 9,000 (5 + 4 + 3 + 2)] \times 15}{30} = + 112,500$
<i>g h</i>	$\frac{[58,500 \times 8 - 9,000 (7 + 6 + 5 + 4 + 3 + 2)] \times 15}{30} = + 112,500$
<i>C E</i>	$\frac{(58,500 \times 2 - 9,000 \times 1) \times 15}{30} = - 54,000$
<i>E G</i>	$\frac{[58,500 \times 4 - 9,000 (3 + 2 + 1)] \times 15}{30} = - 90,000$
<i>G G'</i>	$\frac{[58,500 \times 6 - 9,000 (5 + 4 + 3 + 2 + 1)] \times 15}{30} = - 108,000$

As the total panel load is equal to 18,000 lb. live load plus 9,000 lb. dead load, or 27,000 lb., the maximum chord stresses are equal to the dead-load stresses multiplied by $\frac{27,000}{9,000}$, or 3. They are as follows:

MEMBER	MAXIMUM STRESS, IN POUNDS
<i>a b, b c</i>	$+ 58,500 \times 3 = + 175,500$
<i>c d, d e</i>	$+ 94,500 \times 3 = + 283,500$
<i>e f, f g</i>	$+ 112,500 \times 3 = + 337,500$
<i>g h</i>	$+ 112,500 \times 3 = + 337,500$
<i>C E</i>	$- 54,000 \times 3 = - 162,000$
<i>E G</i>	$- 90,000 \times 3 = - 270,000$
<i>G G'</i>	$- 108,000 \times 3 = - 324,000$

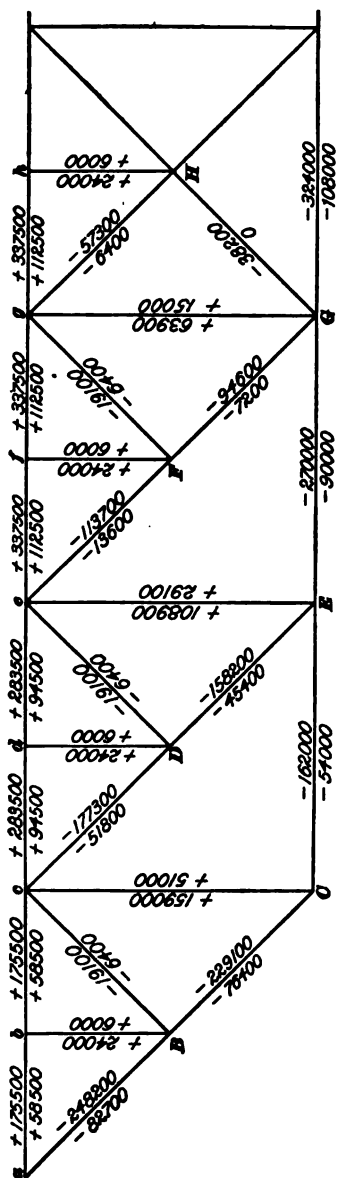


FIG. 37

Web Stresses.—The dead-load shears in the various panels are as given in the following table:

Panel	Dead-Load Shear Pounds	Panel	Dead-Load Shear Pounds
<i>ab</i>	+ 58,500	<i>ef</i>	+ 22,500
<i>bc</i>	+ 49,500	<i>fg</i>	+ 13,500
<i>cd</i>	+ 40,500	<i>gh</i>	+ 4,500
<i>de</i>	+ 31,500		

The maximum positive and negative live-load shears are as follows:

Panel	Positive Live-Load Shear Pounds	Negative Live-Load Shear Pounds
<i>ab</i>	+ 117,000	
<i>bc</i>	+ 100,300	— 1,300
<i>cd</i>	+ 84,900	— 3,900
<i>de</i>	+ 70,700	— 7,700
<i>ef</i>	+ 57,900	— 12,900
<i>fg</i>	+ 46,300	— 19,300
<i>gh</i>	+ 36,000	— 27,000

The maximum and minimum combined shears are as follows:

Panel	Combined Shear	
	Maximum	Minimum
<i>ab</i>	+ 175,500	+ 58,500
<i>bc</i>	+ 149,800	+ 48,200
<i>cd</i>	+ 125,400	+ 36,600
<i>de</i>	+ 102,200	+ 23,800
<i>ef</i>	+ 80,400	+ 9,600
<i>fg</i>	+ 59,800	— 5,800
<i>gh</i>	+ 40,500	— 22,500

From the figure, $\csc H = \frac{\sqrt{30^2 + 30^2}}{30} = 1.414$. The maximum stress in bB , dD , etc. is $18,000 + 6,000 = 24,000$ lb., compression. The minimum stress in bB , dD , etc. is 6,000 lb., compression. The maximum stress in Bc , De , and Fg is

$$\left(\frac{18,000 + 6,000 + 3,000}{2} \right) \times 1.414 = 19,100 \text{ lb., tension}$$

The minimum stress in Bc , De , and Fg is

$$\left(\frac{6,000 + 3,000}{2} \right) \times 1.414 = 6,400 \text{ lb., tension}$$

The maximum and minimum stresses (tension) in aB , cD , eF , and gH are as follows:

MEMBER	STRESS, IN POUNDS	
	MAXIMUM	MINIMUM
aB	$175,500 \times 1.414 = 248,200$	$58,500 \times 1.414 = 82,700$
cD	$125,400 \times 1.414 = 177,300$	$36,600 \times 1.414 = 51,800$
eF	$80,400 \times 1.414 = 113,700$	$9,600 \times 1.414 = 13,600$
gH	$40,500 \times 1.414 = 57,300$	$4,500 \times 1.414 = 6,400$

The maximum and minimum stresses (tension) in BC , DE , FG , and GH are as follows:

MEMBER	MAXIMUM STRESS, IN POUNDS
BC	$(175,500 - 27,000 + 13,500) \times 1.414 = 229,100$
DE	$(125,400 - 27,000 + 13,500) \times 1.414 = 158,200$
FG	$(80,400 - 27,000 + 13,500) \times 1.414 = 94,600$
GH	$(22,500 + 4,500) \times 1.414 = 38,200$
MEMBER	MINIMUM STRESS, IN POUNDS
BC	$(58,500 - 9,000 + 4,500) \times 1.414 = 76,400$
DE	$(36,600 - 9,000 + 4,500) \times 1.414 = 45,400$
FG	$(9,600 - 9,000 + 4,500) \times 1.414 = 7,200$
GH	0

The maximum and minimum stresses (compression) in Cc , Ee , and Gg are as follows:

MEMBER	STRESS, IN POUNDS	
	MAXIMUM	MINIMUM
Cc	$162,000 - 3,000 = 159,000$	$54,000 - 3,000 = 51,000$
Ee	$111,900 - 3,000 = 108,900$	$32,100 - 3,000 = 29,100$
Gg	$66,900 - 3,000 = 63,900$	$4,500 + 6,000 + 4,500 = 15,000$

The maximum and minimum stresses are given in Fig. 37, the former above and the latter below the lines.

EXAMPLE FOR PRACTICE

Let Fig. 34 (a) be a twelve-panel through Baltimore truss having a span length of 216 feet and a height of 36 feet. If the dead load is 1,200 pounds, of which two-thirds is applied at the loaded chord, and

the live load is 2,200 pounds, per linear foot of bridge, find the maximum and minimum combined stresses: (a) in CE , cd , and fg ; (b) in Dd and Bc ; (c) in BC , CD , and EF ; (d) in eF , Ee , and Gg .

	PANEL	STRESS, IN POUNDS	
		MAXIMUM	MINIMUM
Ans. {	(a) { CE	+ 244,800	+ 86,400
	{ cd	- 168,300	- 59,400
	{ fg	- 260,100	- 91,800
	(b) { Dd	- 27,000	- 7,200
	{ Bc	+ 21,600	+ 7,600
	(c) { BC	+ 216,300	+ 76,400
	{ CD	- 136,800	- 38,800
	{ EF	- 66,600	- 7,600
	(d) { eF	+ 7,600	- 8,100
	{ Ee	+ 50,800	+ 9,000
	{ Gg	+ 38,300	+ 3,600

THE FINK AND THE BOLLMAN TRUSS

85. Fig. 38 shows the **Fink truss**, which has been used to some extent for bridge purposes in the past, and is at present employed in a modified form for roof trusses. The analysis of stresses in it should present no difficulty; the method of joints is best adapted to this case. The stress in each short vertical is evidently equal to the panel load at its upper joint. The vertical component of the stress in a short

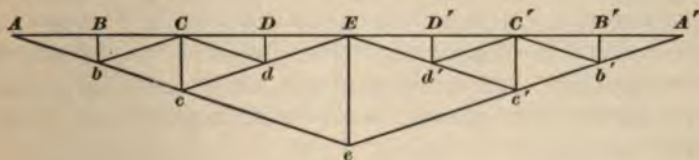


FIG. 38

diagonal, such as Cd , is equal to one-half the sum of the loads at the joints D and d . The stress in the vertical Cc is equal to the sum of the vertical components of the stresses in bC and Cd , and the load at C , etc. The maximum chord stresses obtain when there is a full load; under this condition the horizontal components in bC , dE , and $d'C'$ are equal, respectively, to those in Cd , $E d'$, and $C' b'$, and the compression in the top chord is constant from A to A' and equal to the horizontal component of the stress in $A b$.

86. The **Bollman truss**, shown in Fig. 39 (a), has been used to some extent for bridge purposes in the past, but is now practically obsolete. The load that came on the lower joint of a vertical was carried directly to the ends of the top chord by the two diagonals, shown in full lines, that meet at the bottom of the vertical. The bottom chord and

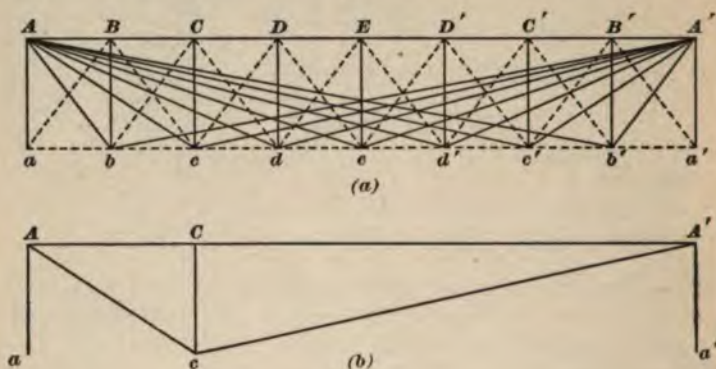


FIG. 39

the dotted diagonals are superfluous members, and were put in to stiffen the truss. As shown in Fig. 39 (b), the vertical components of the stresses in the main diagonals are equal, respectively, to the reactions due to the load that comes to the intersection of any pair. The stress in the top chord is compression, and is equal to the sum of the horizontal components of the stresses in all the main diagonals at one end of the truss.

STRESSES IN BRIDGE TRUSSES

(PART 3)

CURVED- AND INCLINED-CHORD TRUSSES

PRODUCTORY AND GENERAL CONSIDERATIONS

• **Description.**—All the trusses analyzed in *Stresses in Bridge Trusses*, Part 2, have parallel chords. As explained in Part 1, parallel-chord trusses are used for the shorter spans for which trusses are adapted; for the longer spans, curved- and inclined-chord trusses are more economical. In a curved-chord truss, the inclination of the curved chord changes at every panel point; in an inclined-chord truss, the inclination is constant from the center toward the end. In through bridges, the upper chord is curved or inclined; while in deck bridges, the lower, and in some bridges, both chords, are curved or inclined. A length of one or two panels of curved chord near the center of the truss is sometimes horizontal.

• **Stresses in Inclined Members.**—In calculating the stresses in an inclined member, it is convenient to find first the vertical or horizontal component, and calculate the stress from it. The problem frequently arises, therefore, of calculating a stress from one of its components, or one component from the other; for this reason, the relations between a stress and its components, and between the components are stated. They are:

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$S = X \sec H$; $S = Y \csc H$; $X = Y \cot H$; $Y = X \tan H$
in which S = stress;

H = angle that line of action of stress makes with horizontal;

X = horizontal component of stress;

Y = vertical component of stress.

The method of sections and the method by the stress diagram are most useful in finding the stresses in inclined-chord trusses, and they will be used in what follows. The method of joints usually requires more work than either of the other two methods, and will not be used except in special cases.

3. Illustrative Example.—To illustrate the general method of calculating stresses, it will be well to consider a special case, such as the single-system curved-chord truss represented in Fig. 1 (a). If it is desired to find the stress

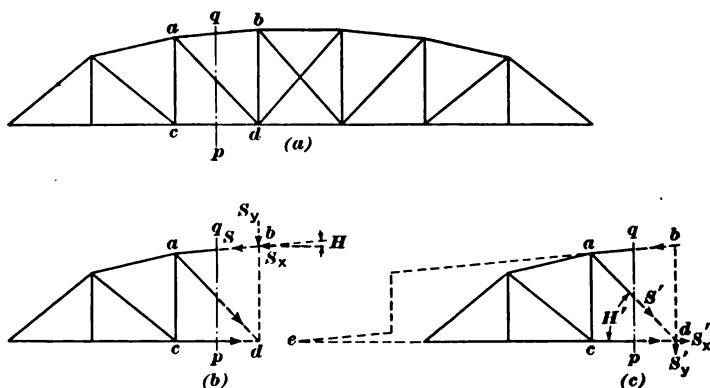


FIG. 1

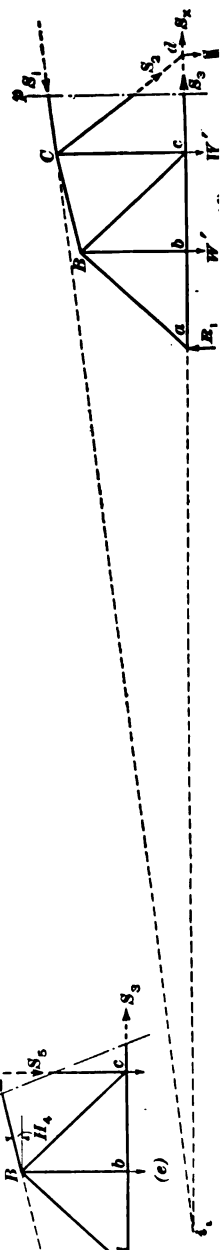
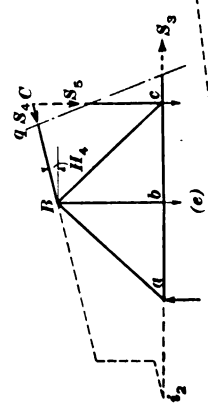
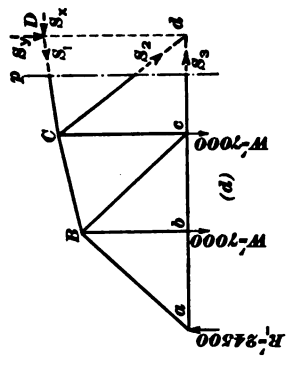
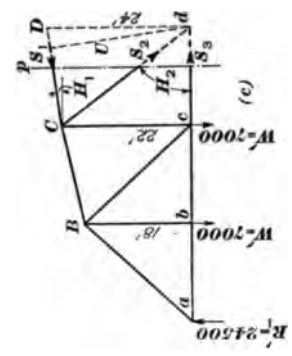
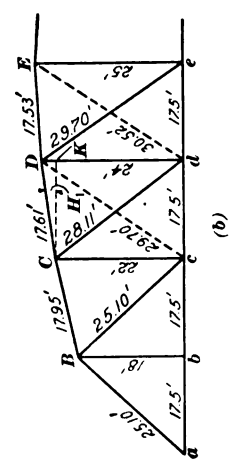
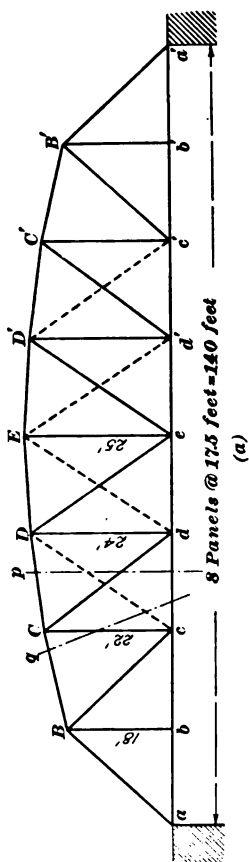
in the members ab , ad , and cd , the truss may be considered cut by the plane of section pq , and the part to the left of this section treated as a free body. Then the stress in the lower-chord member cd is equal to the moment about a of all the other forces acting on the part of the truss to the left of pq divided by the height of the truss at the point a . The stress in the upper-chord member ab is equal to the

bending moment at d divided by the perpendicular distance from d to ab ; in this case, it is more convenient to consider the stress in ab resolved into its vertical and horizontal components S_y and S_x at the point b , as represented in Fig. 1 (b). The moment of S_y about d is equal to zero; then, S_x is equal to the bending moment at the point d divided by the height of the truss at the point d , and S is equal to $S_x \sec H$.

The stress in the web member ad may be found by the method of shears or by the method of moments. As the chord member ab is inclined, the vertical component of the stress in ad is not equal to the shear on section pq , but is equal to the algebraic sum of the shear and the vertical component of the stress in ab due to the same loading. To find the stress in ad by the method of moments, it is convenient to consider it resolved at the point d into its vertical and horizontal components S'_y and S'_x , respectively, as represented in Fig. 1 (c). Then, if moments are taken about the point e , the intersection of ab and cd , the moment of S'_x about e is equal to zero; S'_y is equal to the moment about e of all the other forces acting on the part of the truss to the left of section pq , divided by the distance from d to e ; and S' is equal to $S'_y \csc H'$.

THE CURVED-CHORD TRUSS

4. Description.—Fig. 2 (a) represents the ordinary type of curved-chord truss as used in a through bridge. The members are similar to the corresponding members of the Pratt truss, except that in this case the upper chord is curved. This truss is sometimes spoken of as the **curved-chord Pratt truss**. The method of determining the stresses will be illustrated by considering the eight-panel curved-chord truss represented in Fig. 2 (a). The dead load, all of which will be assumed to be applied at the joints of the loaded chord, is 800 pounds, and the live load 1,800 pounds per linear foot of bridge. The lengths of all the inclined members are shown in Fig. 2 (b).



The dead panel load for one truss is

$$\frac{800 \times 17.5}{2} = 7,000 \text{ pounds,}$$

and each dead-load reaction is equal to

$$\frac{7,000 \times 7}{2} = 24,500 \text{ pounds}$$

The live panel load for one truss is

$$\frac{1,800 \times 17.5}{2} = 15,750 \text{ pounds,}$$

and each reaction for full live load is equal to

$$\frac{15,750 \times 7}{2} = 55,120 \text{ pounds}$$

ANALYTIC METHOD

5. Chord Stresses.—The maximum chord stresses obtain when there is a full live load on the truss; the minimum, when there is no live load. To find the stress in any chord member, such as CD or cd , Fig. 2 (*a*), the truss may be considered cut by a plane at the section p , and the left-hand part treated as a free body, as represented in Fig. 2 (*c*); the forces S_1 , S_2 , S_3 represent the stresses in CD , Cd , and cd , respectively. Then, S_3 is equal to the moment about the point C of all the other forces acting on this part of the truss, divided by the height at c . Therefore, for dead load alone,

$$S_3 = \frac{(24,500 \times 2 - 7,000 \times 1) \times 17.5}{22} = -33,400 \text{ pounds,}$$

using the minus sign to indicate tension.

The force S_1 may be considered replaced by its components S_y and S_x at D , Fig. 2 (*d*). The moment of S_y about d is equal to zero; S_x is equal to the bending moment at d divided by the height at d . Therefore, for dead load alone,

$$S_x = \frac{[24,500 \times 3 - 7,000 (2 + 1)] \times 17.5}{24} = 38,300 \text{ pounds}$$

For CD , the angle $H_1 = DCK$, Fig. 2 (*b*), and

$$\begin{aligned} S_1 &= S_x \sec DCK = S_x \times \frac{CD}{CK} = 38,300 \times \frac{17.61}{17.5} \\ &= +38,500 \text{ pounds} \end{aligned}$$

In like manner, the stress in any chord member may be found.

If the truss is considered cut by an oblique plane that intersects two chord members and a vertical, and the part to the left of the section is considered as a free body, as represented in Fig. 2 (*e*), it will be seen that the only horizontal forces acting on this part are the stress in *cd* and the horizontal component of the stress in *BC*. Therefore, these two forces are numerically equal. In like manner, it may be shown that the stress in *de* is numerically equal to the horizontal component of the stress in *CD*.

The dead-load chord stresses are the minimum chord stresses. They are as follows, expressed in pounds:

Stress in *ab* and *bc*,

$$\frac{(24,500 \times 1) \times 17.5}{18} = -23,800 \checkmark$$

Stress in *cd*,

$$\frac{(24,500 \times 2 - 7,000 \times 1) \times 17.5}{22} = -33,400 \checkmark$$

Stress in *de*,

$$\frac{[24,500 \times 3 - 7,000 \times (2 + 1)] \times 17.5}{24} = -38,300 \checkmark$$

Stress in *BC*,

$$33,400 \times \frac{17.95}{17.5} = +34,300$$

Stress in *CD*,

$$38,300 \times \frac{17.61}{17.5} = +38,500$$

Stress in *DE*,

$$\frac{[24,500 \times 4 - 7,000 \times (3 + 2 + 1)] \times 17.5}{25} \times \frac{17.53}{17.5} = +39,300$$

The live-load chord stresses are found from the above by multiplying them by $\frac{18,900}{8000}$, or $\frac{9}{4}$. They are as follows:

MEMBER	STRESS, IN POUNDS	MEMBER	STRESS, IN POUNDS
<i>ab, bc</i>	-53,600	<i>BC</i>	+77,200
<i>cd</i>	-75,200	<i>CD</i>	+86,600
<i>de</i>	-86,200	<i>DE</i>	+88,400

The maximum chord stresses are found by adding the dead- and live-load chord stresses. They are as follows:

MEMBER	MAXIMUM STRESS, IN POUNDS	MEMBER	MAXIMUM STRESS, IN POUNDS
<i>ab, bc</i>	— 77,400	<i>BC</i>	+ 111,500
<i>cd</i>	— 108,600	<i>CD</i>	+ 125,100
<i>de</i>	— 124,500	<i>DE</i>	+ 127,700

6. Web Stresses in General.—The web stresses may be found by the method of shears or by the method of moments. Both methods will be explained; but, for the actual determination of stresses, the shorter method will be used in each case.

Method of Shears.—Applying the equation $\Sigma Y = \Sigma S \sin H = 0$ to the forces acting on the part of the truss represented in Fig. 2 (*c*), we have

$$\Sigma Y = R_1' - W' - W' - S_1 \sin H_1 - S_2 \sin H_2 = 0;$$

whence

$$S_2 \sin H_2 = R_1' - W' - W' - S_1 \sin H_1 \quad (1)$$

Likewise, applying the same equation to the part shown in Fig. 2 (*e*),

$$\Sigma Y = R_1' - W' - W' - S_2 \sin H_2 - S_3 = 0;$$

whence

$$S_3 = R_1' - W' - W' - S_2 \sin H_2 \quad (2)$$

In equations (1) and (2), $R_1' - W' - W'$ is the shear on section *p* and on section *q*; $S_1 \sin H_1$ and $S_2 \sin H_2$ are the vertical components of the stresses in *CD* and *BC*, respectively; $S_2 \sin H_2$ is the vertical component of the stress in the diagonal *Cd*; and S_3 is the stress in the vertical *Cc*. Then, the vertical component of the stress in any intermediate diagonal, and the stress in any intermediate vertical, of a curved-chord truss is equal to the algebraic sum of the shear on the plane of section that intersects the web member under consideration and two chord members, and the vertical component of the stress in the inclined chord member intersected by the plane. The chord stress referred to is that which obtains for the same loading that causes the desired web stress.

Method of Moments.—To find the stress in Cd by the method of moments, the equation $\Sigma M = 0$ is applied to the forces acting on the part of the truss represented in Fig. 2 (*c*), taking for the center of moments the point of intersection i_1 of the lines of action of S_1 and S_2 . The work may be shortened by assuming that S_2 is replaced at the point d by its vertical and horizontal components S_y and S_x , respectively, as represented in Fig. 2 (*f*). The moments of S_1 , S_y , and S_x about i_1 are each equal to zero; then, S_y is equal to the moment about i_1 of all the external forces acting on the part shown, divided by the distance from d to i_1 . This distance is calculated as follows: CD slopes 2 feet vertical in one panel, or in 17.5 feet horizontal; then, since the height of the truss at D is 24 feet, the point of intersection of the lines of action of S_1 and S_y is $24 \div 2 = 12$ panel lengths to the left of Dd . Then, i_1 is 11×17.5 feet to the left of c , 10×17.5 feet to the left of b , and 9×17.5 feet to the left of a . Applying the equation $\Sigma M = 0$, we have

$$\Sigma M = (R_1' \times 9 - W' \times 10 - W' \times 11) \times 17.5 - S_y \times 12 \times 17.5 = 0;$$

whence

$$\begin{aligned} S_y &= \frac{(R_1' \times 9 - W' \times 10 - W' \times 11) \times 17.5}{12 \times 17.5} \\ &= \frac{R_1' \times 9 - W' \times (10 + 11)}{12} \end{aligned}$$

and $S_x = S_y \csc H_2$.

To find the stress in Cc by the method of moments, the equation $\Sigma M = 0$ is applied to all the forces acting on the part of the truss represented in Fig. 2 (*e*), the center of moments being taken at i_2 , the intersection of BC and cd produced; i_2 is found to be 3.5×17.5 feet to the left of a . Applying the equation $\Sigma M = 0$, we have

$$\Sigma M = (R_1' \times 3.5 - W_1' \times 4.5 - W_1' \times 5.5) \times 17.5 - S_x \times 5.5 \times 17.5 = 0;$$

whence
$$S_x = \frac{R_1' \times 3.5 - W_1' (4.5 + 5.5)}{5.5}$$

From the foregoing, the following general principle may be stated:

The vertical component of the stress in an intermediate diagonal, or the stress in an intermediate vertical of a curved-chord truss is equal to the moment of the external forces on the left of the plane of section that cuts the web member and two chord members, about the point of intersection of the chord members cut by the plane, divided by the distance from this intersection to that of the web member under consideration with the horizontal chord.

7. The method of moments is especially useful in the calculation of the maximum live-load stresses, as the left reaction is then usually the only external force to the left of the section. The method of shears is best suited to the calculation of the dead-load stresses, as the horizontal components of the chord stresses are usually known before the stresses in the web members are found, and the vertical components may be found from them.

8. **Dead-Load Web Stresses.**—The dead-load web stresses will be found by the method of shears. The

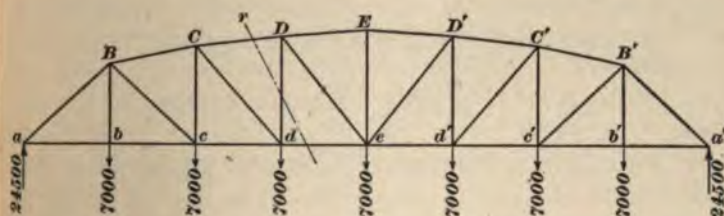


FIG. 3

members in action, and the loads and reactions, are shown in Fig. 3. The shears are as follows:

PANEL	SHEAR, IN POUNDS	PANEL	SHEAR, IN POUNDS
<i>a b</i>	+ 24,500	<i>c d</i>	+ 10,500
<i>b c</i>	+ 17,500	<i>d e</i>	+ 3,500
		<i>e d'</i>	— 3,500

The vertical components of the chord stresses are found from the horizontal components given in Art. 5. They are as follows:

MEMBER	VERTICAL COMPONENT OF STRESS, IN POUNDS
<i>BC</i>	$33,400 \times \frac{4}{17.5} = 7,600$
<i>CD</i>	$38,300 \times \frac{2}{17.5} = 4,400$
<i>DE</i>	$39,200 \times \frac{1}{17.5} = 2,200$

Then, the vertical components of the stresses in the main diagonals are as follows:

MEMBER	VERTICAL COMPONENT, IN POUNDS
<i>aB</i>	24,500
<i>Bc</i>	$17,500 - 7,600 = 9,900$
<i>Cd</i>	$10,500 - 4,400 = 6,100$
<i>De</i>	$3,500 - 2,200 = 1,300$

The actual stresses in the main diagonals are, therefore:

MEMBER	STRESS, IN POUNDS
<i>aB</i>	$24,500 \times \frac{25.10}{18} = + 34,200$
<i>Bc</i>	$9,900 \times \frac{25.10}{18} = - 13,800$
<i>Cd</i>	$6,100 \times \frac{28.11}{22} = - 7,800$
<i>De</i>	$1,300 \times \frac{29.70}{24} = - 1,600$

The stresses in the verticals are as follows:

MEMBER	STRESS, IN POUNDS
<i>Bb</i>	$W = - 7,000$
<i>Cc</i>	$10,500 - 7,600 = + 2,900$
<i>Dd</i>	$3,500 - 4,400 = - 900$
<i>Ee</i>	$2,200 + 2,200 = - 4,400$

The stress in *Dd* is tension, because the vertical component of the stress in *CD*, which acts downwards on the

part of the truss to the left of section r , Fig. 3, is greater than the shear on section r ; therefore, the stress in Dd must act upwards from the joint d , and is tension. The stress in Ee is equal to the sum of the vertical components of the stresses in DE and ED' , both of which act upwards at the joint E ; therefore, the stress in Ee must act downwards from E , and is tension.

9. Counters.—When the counter dE is in action, the members that are in action are shown in Fig. 4; the main diagonal De is omitted from the diagram, as its stress is then zero. For purposes that will be explained later, it will be convenient here to find the dead-load stresses that would occur in the counter dE and the verticals Dd and Ee , if the main diagonal De were omitted when there is no live load

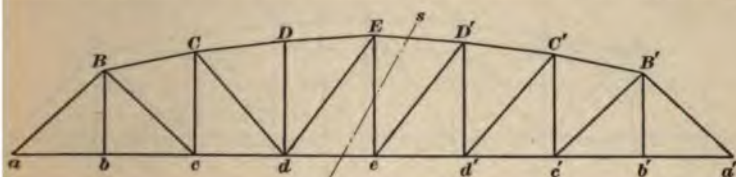


FIG. 4

on the truss. The vertical component of the stress in dE is then equal to the algebraic sum of the shear in panel de and the vertical component of the stress in DE . The horizontal component of the stress in DE under these conditions is equal to the moment at d divided by the height at d ; this is the same as the horizontal component of the stress in CD found in Art. 5, or 38,300 pounds. Then, the vertical component of the stress in DE , under these conditions, is equal to

$$38,300 \times \frac{1}{17.5} = 2,190 \text{ pounds}$$

The shear in panel de is 3,500 pounds; therefore, the vertical component of the stress in dE is $3,500 - 2,190 = 1,310$ pounds; and the stress in dE is

$$1,310 \times \frac{30.52}{25} = +1,600 \text{ pounds}$$

The stress in Dd is equal to the algebraic sum of the vertical components in CD and DE ; therefore, the stress in Dd is

$$38,300 \times \frac{2}{17.5} - 38,300 \times \frac{1}{17.5} = -2,190 \text{ pounds}$$

The stress in Ee is equal to the algebraic sum of the shear on section s and the vertical component of the stress in $D'E$. The shear on the section s is $+3,500$ pounds; the vertical component of the compression in $D'E$ is $2,200$ pounds (see Art. 8). Then, the stress in Ee is $3,500 + 2,200 = -5,700$ pounds.

When the counter cD is in action, the members that are in action are shown in Fig. 5; the main diagonals Cd and De are omitted, as the stresses in them are then zero. It will be convenient here to find the dead-load stresses that would

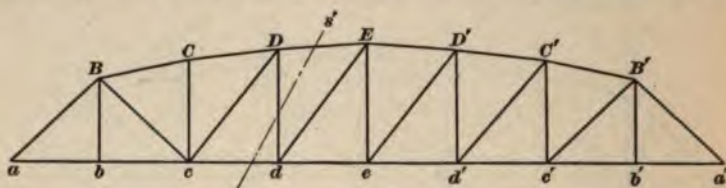


FIG. 5

occur in the counter cD and in the verticals Cc and Dd , if the main diagonals were omitted when there is no live load on the truss. The vertical component of the stress in cD is equal to the algebraic sum of the shear in panel cd and the vertical component of the stress in CD . The center of moments for CD is now c , and the horizontal component of the stress in CD is equal to the horizontal component of the stress in BC as found in Art. 5, or $33,400$ pounds. Then, the vertical component in CD is equal to

$$33,400 \times \frac{2}{17.5} = 3,800 \text{ pounds}$$

The shear in the panel cd is $10,500$ pounds; therefore, the vertical component of the stress in cD is $10,500 - 3,800 = 6,700$ pounds, and the stress in cD is

$$6,700 \times \frac{29.70}{24} = +8,300 \text{ pounds}$$

The stress in Cc is equal to the algebraic sum of the vertical components in BC and CD ; that is, the stress in Cc is

$$33,400 \times \frac{4}{17.5} - 33,400 \times \frac{2}{17.5} = -3,800 \text{ pounds}$$

The stress in Dd is equal to the algebraic sum of the shear on section s' and the vertical component of the stress in DE . The vertical component of the stress in DE is 2,190 pounds. The shear on section s' is 10,500 pounds; therefore, the stress in Dd is $10,500 - 2,190 = -8,310$ pounds.

It must be understood that *the compressive stresses in the counters given above do not really exist for any loading; they are the amounts by which the live-load tensions in these members must be reduced in order to get the actual tensions in the counters.* If the live-load tension in any counter is less numerically than the dead-load compression, that means that the counter is not required.

10. Live-Load Web Stresses.—The positions of live load that cause maximum web stresses are in general the same as in the Pratt truss.

For example, the maximum stresses in Cd and Cc , Fig. 6,

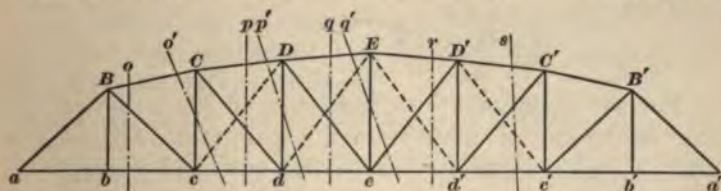


FIG. 6

occur when all joints from d to b' are loaded; the maximum stresses in De and Dd occur when all joints from e to b' are loaded; etc.

The live-load web stresses will now be found. The maximum live-load stress in aB is equal to the shear in panel ab multiplied by $\csc H$, or

$$\frac{15,750 \times 7}{2} \times \csc H = 55,120 \times \frac{25.10}{18} = +76,900 \text{ pounds.}$$

The maximum live-load stress in Bb is a live panel load, or $-15,750$ pounds. The method of moments will be used

in calculating the remaining web stresses. The distances to the points of intersection of the various upper-chord members with the lower chord are as follows:

For BC , 3.5 panels to the left of a .

For CD , 9 panels to the left of a .

For DE , 21 panels to the left of a .

For ED' , 21 panels to the right of a' .

For $D'C'$, 9 panels to the right of a' .

For the member Bc , section o , Fig. 6, with loads from c to b' , the center of moments is 3.5 panels to the left of a . Under this condition of loading,

$$R_1'' = \frac{15,750 \times (1 + 2 + 3 + 4 + 5 + 6)}{8} = 41,300 \text{ pounds}$$

and the stress in Bc is

$$\frac{41,300 \times 3.5}{5.5} \times \frac{25.10}{18} = 26,300 \times \frac{25.10}{18} = -36,700 \text{ pounds}$$

For the member Cc , section o' , with loads from d to b' , the center of moments is 3.5 panels to the left of a . Here,

$$R_1'' = \frac{15,750 \times (1 + 2 + 3 + 4 + 5)}{8} = 29,500 \text{ pounds}$$

and the stress in Cc is

$$\frac{29,500 \times 3.5}{5.5} = +18,800 \text{ pounds}$$

For the member Cd , section p , with loads from d to b' , the center of moments is nine panels to the left of a , and $R_1'' = 29,500$ pounds. Then, the stress in Cd is

$$\frac{29,500 \times 9}{12} \times \frac{28.11}{22} = 22,100 \times \frac{28.11}{22} = -28,200 \text{ pounds}$$

For the member Dd , section p' , with loads from e to b' , the center of moments is nine panels to the left of a , and $R_1'' = 19,700$ pounds. Then, the stress in Dd is

$$\frac{19,700 \times 9}{12} = +14,800 \text{ pounds}$$

For the member De , section q , with loads from e to b' , the center of moments is twenty-one panels to the left of a , and $R_1'' = 19,700$ pounds. Then, the stress in De is

$$\frac{19,700 \times 21}{25} \times \frac{29.70}{24} = 16,500 \times \frac{29.70}{24} = -20,400 \text{ pounds}$$

For the member Ee , the stress is a maximum when the loads extend from d' to b' and the counter Ed' is in action. Then (section q'), the center of moments is twenty-one panels to the left of a ; $R_1'' = 11,800$ pounds; and, therefore, the stress in Ee is

$$\frac{11,800 \times 21}{25} = +9,900 \text{ pounds}$$

For the counter Ed' , section r , with loads from d' to b' , the center of moments is twenty-one panels to the right of a' , that is, twenty-nine panels to the right of a ; $R_1'' = 11,800$ pounds; and the stress in Ed' is

$$\frac{11,800 \times 29}{24} \times \frac{30.52}{25} = 14,300 \times \frac{30.52}{25} = -17,500 \text{ pounds}$$

For the counter $D'e'$, section s , with loads at c' and b' , the center of moments is nine panels to the right of a' , that is, seventeen panels to the right of a ; $R_1'' = 5,900$ pounds; and the stress in $D'e'$ is

$$\frac{5,900 \times 17}{11} \times \frac{29.70}{24} = -11,300 \text{ pounds}$$

to here
11. Maximum Web Stresses.—Combining the maximum live-load stresses with the dead-load stresses, the following results are obtained:

Member	Dead-Load Stress	Maximum Live-Load Stress	Combined Maximum Stress
aB	+ 34,200	+ 76,900	+ 111,100
Bb	— 7,000	— 15,800	— 22,800
Bc	— 13,800	— 36,700	— 50,500
Cc	+ 2,900	+ 18,800	+ 21,700
Cd	— 7,800	— 28,200	— 36,000
Dd	— 900	+ 14,800	+ 13,900
Dc	— 1,600	— 20,400	— 22,000
Ee	— 5,700	+ 9,900	+ 4,200
dE or Ed'	+ 1,600	— 17,500	— 15,900
cD or $D'e'$	+ 8,300	— 11,300	— 3,000

As the resulting combined stresses in dE and cD are tension, both these counters are required. The dead-load stress given for Ee is the stress found on the supposition that the main diagonal in panel de or $d'e$ is omitted from the truss. As the main diagonal is out of action for the loading causing the maximum live-load stress in Ee , the dead-load value given in the table is the one to use in obtaining the combined stress.

12. Minimum Web Stresses.—The minimum stress in the end post aB is equal to the dead-load stress of 34,200 pounds, compression. For the minimum stress in Bc , the joint b should be loaded. The center of moments for Bc , as before, is 3.5 panels to the left of a ; the live-load reaction for the load at b is

$$R_1'' = \frac{15,750 \times 7}{8} = 13,800 \text{ pounds}$$

and the live-load stress in Bc is

$$\frac{13,800 \times 3.5 - 15,750 \times 4.5}{5.5} \times \frac{25.10}{18} = -4,100 \times \frac{25.1}{18} \\ = -5,700 \text{ pounds}$$

In writing this equation, the stress in Bc was assumed as tension; the negative sign of the result indicates that the live load tends to cause compression in Bc . The dead-load stress in Bc is 13,800 pounds, tension; therefore, the actual stress in Bc for this loading is $13,800 - 5,700 = 8,100$ pounds tension, which is the minimum stress in Bc . The minimum stress in each of the remaining main diagonals and in the counters is zero; when one diagonal in any panel is in action, the other is assumed to be idle.

13. Determination of Panels in Which Counters Are Required.—It should be noted that it is not always possible, in a curved- or inclined-chord truss, to determine directly from the shears the panels in which counters are required. In the analysis of the parallel-chord Pratt truss in *Stresses in Bridge Trusses*, Part 2, it was explained that counters are required only in those panels in which the minimum combined shears are of an opposite kind to the

maximum. If this rule is applied in the present case, it will be seen that the result is not correct. In the panel cd , for example, the maximum shear occurs when all the joints from d to b' are loaded with the live load, and is equal to the sum of 10,500 pounds dead, and 29,500 pounds live, or 40,000 pounds combined positive shear; the minimum shear occurs when the joints b and c are loaded, and is equal to 10,500 pounds dead, and - 5,900 pounds live, or 4,600 pounds combined positive shear. The minimum shear being of the same sign as the maximum, seems to indicate that no counter is required, although it was shown in Art. 11 that a counter is required in the panel cd . No mistake can be made, however, if the minimum stress in each main diagonal is calculated: if the minimum stress comes out tension, no counter is required; if it comes out compression, a counter is required. For example, if no counter were required in the panel cd , the minimum tension in the main diagonal Cd would occur when the joints b and c are loaded with the live load, the left reaction being $\frac{15,750 \times (6+7)}{8} = 25,600$ pounds. The center

of moments for Cd is nine panels to the left of a ; then the live-load stress in Cd would be equal to

$$\frac{25,600 \times 9 - 15,750 \times 10 - 15,750 \times 11}{12} \times \frac{28.11}{22}$$

$$= - 10,700 \text{ pounds}$$

This equation was written assuming the stress in Cd as tension; the negative sign of the result indicates that the live loads at b and c tend to cause compression in Cd . The dead-load stress in Cd is 7,800 pounds tension. Then, the combined stress in Cd would be equal to $10,700 - 7,800 = 2,900$ pounds, compression; but, as no compression can exist in this member, the counter cD is required.

14. Verticals.—The minimum stress in Bb is equal to a dead panel load, which is 7,000 pounds, tension. The minimum stress in each of the other verticals Cc , Dd , and Ee occurs when the stress in the diagonal that meets the vertical at its upper joint is a minimum, that is, when the

two diagonals that meet the vertical at its lower joint are in action. For this condition, the stress in the vertical is tension, and is equal to the algebraic sum of the vertical components of the stresses in the two chord members that meet the vertical at its upper joint. As the minimum stress is of an opposite kind to the maximum, it is necessary to find the maximum tension in the verticals.

The maximum tension in Cc occurs when the diagonals Bc and cD are in action. The members of the truss that are then in action are shown in Fig. 5. The center of moments for both BC and CD is at c ; therefore, the horizontal component in each of them is equal to the moment at c divided by 22. The tension in Cc is equal to the vertical component in BC , which acts upwards, minus the vertical component in CD , which acts downwards; then, the minimum live-load stress in Cc is

$$\begin{aligned} & \frac{\text{moment at } c}{22} \times \frac{4}{17.5} - \frac{\text{moment at } c}{22} \times \frac{2}{17.5} \\ &= \frac{\text{moment at } c}{22} \times \frac{2}{17.5}, \text{ tension} \end{aligned}$$

For the maximum live-load tension in Cc , the truss should be loaded as fully as possible without bringing the main diagonal Cd into action. When there are live loads at b and c , the total combined tension in the counter cD is equal to 3,000 pounds (Art. 11). A live panel load at d would tend to cause compression in cD , that is, decrease this tension by

$$\frac{\frac{15,750 \times 5}{8} \times 9}{11} \times \frac{29.70}{24} = 10,000 \text{ pounds}$$

As this is greater than the tension in cD , the live load cannot be applied at d , as then the main diagonal Cd would be in action. The left reaction for loads at b and c is equal to 25,600 pounds. Then, the live-load moment at c is

$$(25,600 \times 2 - 15,750 \times 1) \times 17.5$$

and the stress in Cc is

$$\begin{aligned} & \frac{(25,600 \times 2 - 15,750 \times 1) \times 17.5}{22} \times \frac{2}{17.5} \\ &= 3,200 \text{ pounds, tension} \end{aligned}$$

The dead-load stress in Cc when the main diagonal Cd is left out was found in Art. 9, and is 3,800 pounds, tension. Then, the maximum tension, or minimum stress, in Cc is equal to $3,200 + 3,800 = 7,000$ pounds. The reason for finding the stresses discussed in Art. 9 is now apparent.

The maximum tension in Dd occurs when the diagonals Cd and dE are in action. The members of the truss that are in action are shown in Fig. 4. The center of moments for CD and DE is at d , and the horizontal component of the stress in each of these members is equal to the moment at d divided by 24. The tension in Dd is equal to the vertical component of the stress in CD , which acts upwards, minus the vertical component of the stress in DE , which acts downwards; then, the stress in Dd is

$$\begin{aligned} & \frac{\text{moment at } d}{24} \times \frac{2}{17.5} - \frac{\text{moment at } d}{24} \times \frac{1}{17.5} \\ &= \frac{\text{moment at } d}{24} \times \frac{1}{17.5}, \text{ tension} \end{aligned}$$

It is evident that, for the maximum live-load tension in Dd , the truss should be loaded as fully as possible without bringing the main diagonal De into action. When the live load is applied at b, c , and d , the tension in dE is 15,900 pounds (Art. 11), and it was shown in the preceding paragraph that under this loading the main diagonal Cd is in action. A live panel load at e decreases the tension in dE by

$$\left(\frac{15,750 \times 4}{8} \times \frac{21}{24} \right) \times \frac{30.52}{25} = 8,400 \text{ pounds}$$

leaving $15,900 - 8,400 = 7,500$ pounds, tension, in dE . A live load at d' further decreases this tension by

$$\left(\frac{15,750 \times 3}{8} \times \frac{21}{24} \right) \times \frac{30.52}{25} = 6,300 \text{ pounds}$$

leaving $7,500 - 6,300 = 1,200$ pounds, tension, in dE . A live load at e' further decreases this tension by

$$\left(\frac{15,750 \times 2}{8} \times \frac{21}{24} \right) \times \frac{30.52}{25} = 4,200 \text{ pounds}$$

As this result is greater than the 1,200 pounds, tension, in dE , the main diagonal De would be in action if the live

load extended to c' . The joints, therefore, from b to d' may be loaded, and still have the counter dE in action. Then,

$$R_1'' = \frac{15,750 \times (3 + 4 + 5 + 6 + 7)}{8} = 49,200 \text{ pounds}$$

and the live-load stress in Dd is equal to

$$\frac{[49,200 \times 3 - 15,750 \times (2 + 1)] \times 17.5}{24} \times \frac{1}{17.5} \\ = 4,200 \text{ pounds, tension}$$

The dead-load stress in Dd , when the main diagonal De is omitted, was found in Art. 9 to be 2,200 pounds. Then, the total maximum tension (minimum stress) in Dd is equal to $4,200 + 2,200 = 6,400$ pounds.

The maximum tension in Ee occurs when the main diagonals De and $D'e$ are in action. The center of moments for both DE and ED' is at e , and the horizontal component of the stress in each of these members is equal to the moment at e divided by 25. The tension in Ee is equal to the sum of the vertical component of the stresses in these two members, as both act upwards at the joint E . The stress in Ee is, therefore,

$$\frac{\text{moment at } e}{25} \times \frac{1}{17.5} + \frac{\text{moment at } e}{25} \times \frac{1}{17.5} \\ = \frac{\text{moment at } e}{25} \times \frac{2}{17.5}, \text{ tension}$$

The maximum moment at e occurs when there is a full load on the truss. The live-load stress in Ee is, then,

$$\frac{[55,120 \times 4 - 15,750 \times (3 + 2 + 1)] \times 17.5}{25} \times \frac{2}{17.5} \\ = 10,100 \text{ pounds, tension}$$

The dead-load stress in Ee , when the main diagonals eD and eD' are in action, is equal to 4,400 pounds, tension. Then, the total maximum tension in Ee is $10,100 + 4,400 = 14,500$ pounds.

In this case, the tension of 14,500 pounds in Ee , found under the head of minimum stresses, is numerically greater than the compression of 4,200 pounds, found under the head of maximum stresses; so that the former is really the maximum. In all the verticals, except the hip vertical, the

minimum stress is of opposite kind to the maximum, and they must be designed to resist both kinds of stress. In a longer truss, or in one with different proportions, there would probably be more than the hip vertical in which the maximum and minimum stresses would be of the same kind. For example, if the counter cD were not required, the minimum compression in Cc would be equal to the algebraic sum of the minimum positive shear in the panel cd or on the plane of sections q , Fig. 2 (*a*), and the vertical component of the stress in BC .

The maximum and minimum stresses are given in Fig. 7,

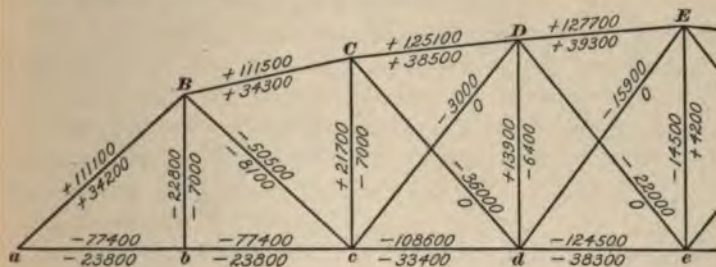


FIG. 7

the former above and the latter below the lines representing the members.

15. Distribution of Dead Load.—In case it is desired to distribute the dead load between the two chords, one-third may be assumed to be applied at the joints of the unloaded chord. The stresses in the verticals only will be slightly different from what they are when all the dead load is assumed to be applied at the joints of the loaded chord.

GRAPHIC METHOD

16. Dead-Load Stresses.—The stress diagram for dead load is represented in Fig. 8 (*b*); the truss, numbered according to Bow's notation, is shown in Fig. 8 (*a*). As the truss is symmetrical about the center, and therefore the stresses in the corresponding members on the two sides of

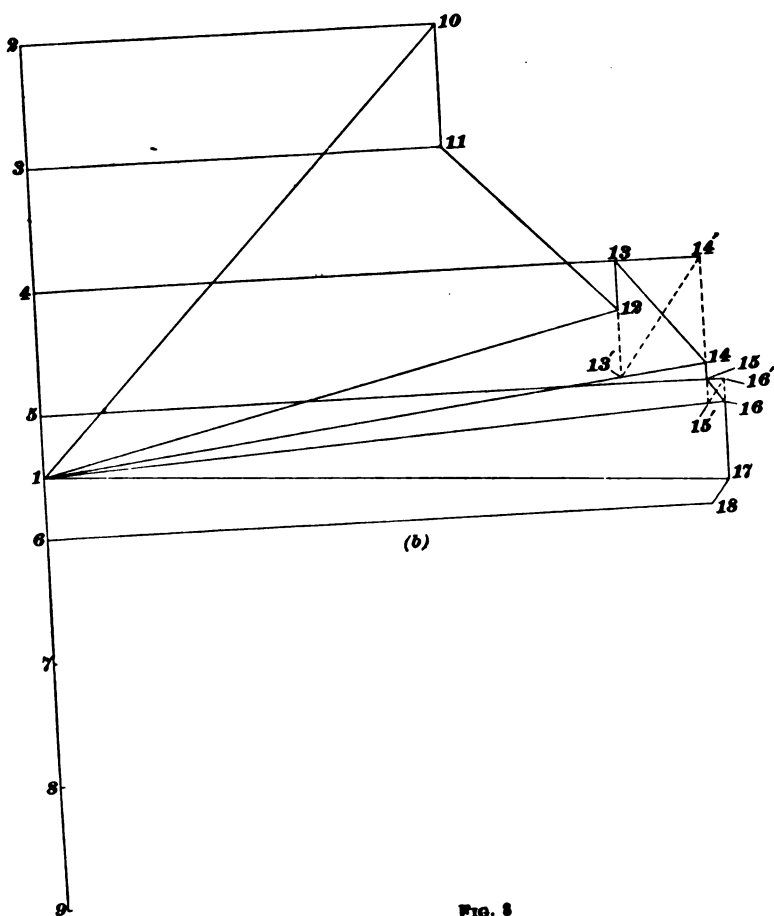
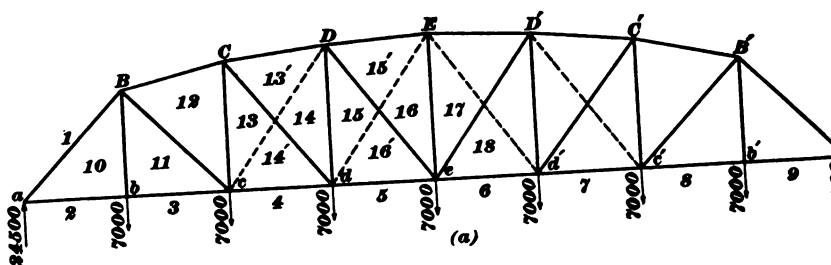


FIG. 8

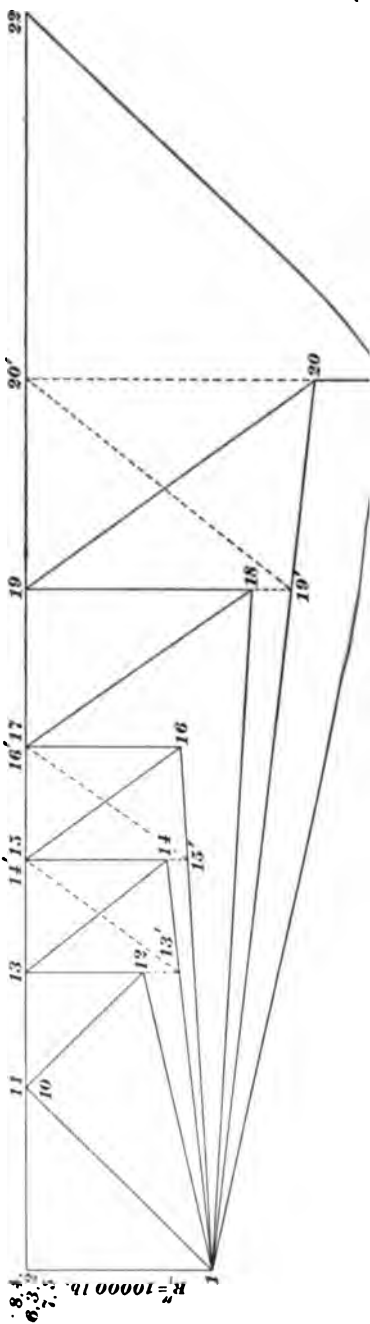
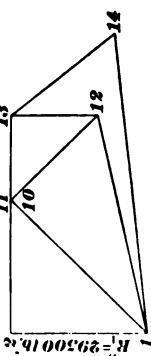
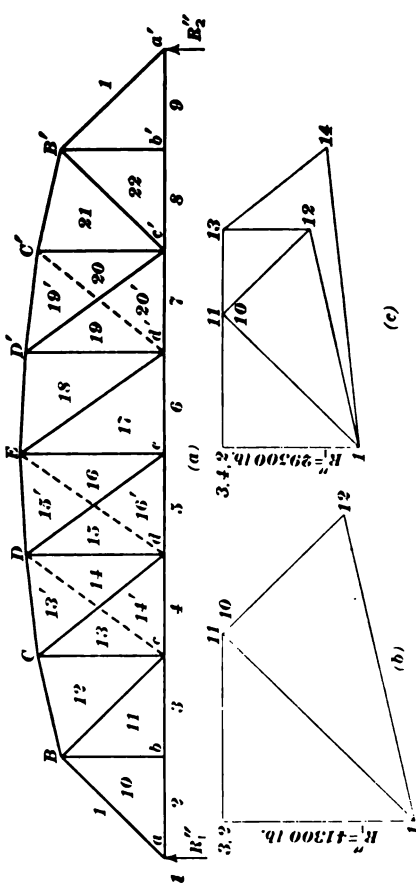
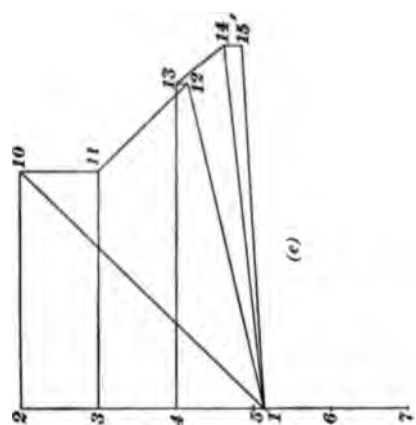
the center are the same, the stress diagram has been drawn for the left end only, although the load line has been laid off for the whole truss. The left reaction 1-2 was first laid off, then the panel loads 2-3, 3-4, etc., and finally the right reaction 9-1. The stress diagram was first drawn for the joint *a*, by drawing 1-10 parallel to *aB*, and 2-10 parallel to *ab*. The vectors 2-10 and 10-1 give the stresses in *ab* and *aB*, respectively. The diagram for the joint *b* was then drawn by drawing through 3 the line 3-11 parallel to *bc*, and through 10 the line 10-11 parallel to *Bb*. Next, the diagram for joint *B*, then that for joint *c*, etc. were drawn in the manner explained in *Graphic Statics*.

The full lines in the stress diagram represent the dead-load stresses when the main diagonals are in action; the dotted lines represent the dead-load counter stresses. In drawing the vectors corresponding to the latter, counters were assumed in the panels in which they were most likely to be needed, and the counter stresses found by assuming that the main diagonals in these panels were left out. For example, if the main diagonal *De* is left out, the diagram for joint *D* is 1-14-15'-1; for joint *d*, 13-4-5-16'-15'-14-13; and for joint *E*, 1-15'-16'-17-1. If the main diagonal *Cd* is left out, the diagram for joint *C* is 1-12-13'-1; for joint *c*, 12-11-3-4-14'-13'-12; for joint *D*, 1-13'-14'-15'-1; and for joint *d*, 14'-4-5-16'-15'-14'.

The characters and values of the stresses may be found from the stress diagram in the usual way; it is unnecessary to tabulate them here, as they are the same as found in the preceding articles.

17. Live-Load Chord Stresses.—As the maximum live-load stresses in the chords obtain when there is a full live load on the truss, they may be found by multiplying the dead-load stresses by the ratio of the live to the dead load, or a stress diagram similar to that shown in Fig. 8 (*b*) may be drawn, the dotted lines being omitted.

18. Live-Load Web Stresses.—The maximum live-load stress in the end post may be found in the same way as



the live-load chord stresses; the maximum live-load stress in the hip vertical is tension and equal to a live panel load. In order to find the live-load stress in any other web member, it is necessary to consider the loading that causes the desired stress in the member, and the members of the truss that are in action for that loading. The maximum live-load stress in any main member on the left of the center obtains when the joints to the right of that member are loaded; the maximum stress in a counter on the left of the center obtains when the joints to the left of the member are loaded. As will be explained presently, it is more convenient to find the maximum stresses in the counters on the right of the center. The only external force that acts on the left of a web member when the live-load stress in that member is greatest is then the left reaction, which can be found most easily by calculation. Each member must receive separate consideration.

The maximum stress in Bc obtains when the truss is loaded from b' to c , Fig. 9 (*a*); then, $R_1'' = 41,300$ pounds. It is unnecessary in this case to lay off the load line for all the external forces acting on the truss; the left reaction 1-2 may be laid off, Fig. 9 (*b*), and the stresses in the members that meet at a found by drawing the stress diagram for the joint a , as represented in Fig. 9 (*b*); then the stress diagram may be drawn for the joints b and B until 11-12, representing the stress in Bc , is found. The maximum stresses in Cc and Cd obtain when all the joints from b' to d are loaded; then, $R_1'' = 29,500$ pounds. This may be laid off, as shown at 1-2, Fig. 9 (*c*), and the stress diagram drawn for the joints a , b , B , c , and C until 12-13 and 13-14, representing the stresses in Cc and Cd , are found.

As the vectors in Fig. 9 (*c*), as far as point 12, are parallel, respectively, to those in Fig. 9 (*b*), the two figures are similar; therefore,

$$\frac{(11-12)_b}{(11-12)_c} = \frac{(1-2)_b}{(1-2)_c} = \frac{41300}{29500};$$

whence $(11-12)_b = \frac{41300}{29500} \times (11-12)_c$

This principle is made use of in Fig. 9 (*d*); the force 1-2, representing the left reaction, was laid off arbitrarily equal to 10,000 pounds, and the stress diagram drawn in the same way as in Fig. 9 (*b*) until 21-22, representing the minimum live-load stress in $B'c'$, was found. In drawing the diagram, it was assumed that a counter would be required in the panel $e d'$, so the main diagonal $e D'$ was left out; in the panel $d' c'$, the stress diagram was drawn first on the assumption that the main diagonal $d' C'$ was left out, and then on the assumption that the counter $D' c'$ was left out. The vector 19-20 represents the stress in $D' c'$, and 19'-20' represents the stress in $d' C'$; the vectors 13'-14' and 15'-16' were drawn in connection with the minimum stresses in the verticals Cc and Dd , as will be explained later. It is evident that, as far as point 12, Fig. 9 (*d*) is similar to Fig. 9 (*b*), and, as far as point 14, is similar to Fig. 9 (*c*); whence

$$(11-12)_b = \frac{41800}{10000} \times (11-12)_a$$

$$(13-14)_c = \frac{22500}{10000} \times (13-14)_a$$

from which it follows that the stress in any member of the truss, when there are no loads on the left of the member, may be found from Fig. 9 (*d*), by multiplying the vector that represents the stress in the member by the ratio of the left reaction for the loading that causes the desired stress to 10,000.

The stresses in the various web members due to a left reaction of 10,000 pounds are first found by scaling the vectors in Fig. 9 (*d*). They are as follows:

MEMBER	STRESS, IN POUNDS	MEMBER	STRESS, IN POUNDS
Bc	- 8,900	Ee	+ 8,400
Cc	+ 6,400	Ed'	- 14,800
Cd	- 9,600	$D'c'$	- 19,200
Dd	+ 7,500	$d'C'$	+ 18,100
De	- 10,400	$c'B'$	+ 28,500

The left reactions due to the loadings that cause the desired stresses are then found in the same way as in Art. 10. They are as follows:

MEMBER	REACTION, IN POUNDS
<i>Bc</i>	41,300
<i>Cc, Cd</i>	29,500
<i>Dd, De</i>	19,700
<i>Ee, Ed</i>	11,800
<i>D'c'</i>	5,900
<i>d' C'</i>	$15,750 \times \frac{1+2}{8} = 5,900$
<i>c' B'</i>	$15,750 \times \frac{1}{8} = 2,000$

Then, the desired stresses in these members are as follows:

MEMBER	MAXIMUM LIVE-LOAD STRESS, IN POUNDS
<i>Bc</i>	$\frac{41300}{10000} \times 8,900 = - 36,800$
<i>Cc</i>	$\frac{29500}{10000} \times 6,400 = + 18,900$
<i>Cd</i>	$\frac{29500}{10000} \times 9,600 = - 28,300$
<i>Dd</i>	$\frac{19700}{10000} \times 7,500 = + 14,800$
<i>De</i>	$\frac{19700}{10000} \times 10,400 = - 20,500$
<i>Ee</i>	$\frac{11800}{10000} \times 8,400 = + 9,900$
<i>Ed'</i>	$\frac{11800}{10000} \times 14,800 = - 17,500$
<i>D'c'</i>	$\frac{5900}{10000} \times 19,200 = - 11,300$
MEMBER	MINIMUM LIVE-LOAD STRESS, IN POUNDS
<i>d' C'</i>	$\frac{5900}{10000} \times 18,100 = + 10,700$
<i>c' B'</i>	$\frac{2000}{10000} \times 28,500 = + 5,700$

The combined stresses in these members may be found in the same way as in Art. 11.

19. Verticals.—The minimum stresses in *Ee* and *Dd*, and in *Cc* when a counter is needed in panel *cd*, as in this case, require special consideration. The stress in each case is tension, and, as this is of opposite kind to the maximum stress, it is necessary to find the maximum live-load tension in these verticals, in order to find the minimum stresses. For this reason, it is necessary to go through the same operations as in the analytic method to determine the proper loadings.

The maximum tension in Ee occurs when the counters dE and $d'E$ are out of action, and the truss is fully loaded; it may be found from the vector $16-17$ in the stress diagram for dead load, Fig. 8 (*b*), by multiplying the dead-load stress in Ee by the ratio of the live to the dead load.

The maximum tension in Cc occurs when the counter cD is in action (if that counter is required), and the truss is loaded as fully as possible without bringing the main diagonal Cd into action. The counter cD is in action when the joints b and c are loaded, and the stress is evidently equal to the stress in $c'D'$ when the joints b' and c' are loaded. In Fig. 9 (*d*), the vector $19-20$ represents the stress in $c'D'$ for a reaction of 10,000 pounds, and scales 19,200 pounds, tension. The left reaction for live loads at b' and c' is equal to

$$\frac{15,750 \times (1 + 2)}{8} = 5,900 \text{ pounds}$$

Then, the actual live-load stress is

$$\frac{5,900}{19,200} \times 19,200 = -11,300 \text{ pounds}$$

From Fig. 8 (*b*) it may be seen that the dead-load stress in cD (vector $13'-14'$), when the main diagonal Cd is out of action, is equal to 8,300 pounds, compression. Then, the combined stress in cD , when there are live loads at b and c , is equal to $11,300 - 8,300 = 3,000$ pounds, tension. To find whether or not a load at d will bring the main diagonal Cd into action, the vector $13'-14'$, Fig. 9 (*d*), representing the stress in cD , may be considered; this scales 10,200 pounds compression for a left reaction of 10,000 pounds. The left reaction due to a live panel load at d is equal to

$$\frac{15,750 \times 5}{8} = 9,800 \text{ pounds}$$

Then, the actual stress in cD due to the live load at d is equal to

$$\frac{9,800}{10,000} \times 10,000 = +9,800 \text{ pounds}$$

This is greater than the tension in cD , which means that, with live loads at b , c , and d , the counter cD will be out of action. Therefore, the maximum tension in Cc obtains when the live load is applied at b and c , and, for a reaction of

10,000 pounds, is given by the vector 20-21 (5,400 pounds), Fig. 9 (*d*). The actual stress in Cc due to the live load at b and c is then equal to

$$\frac{5,200}{10,000} \times 5,400 = -3,200 \text{ pounds}$$

The dead-load stress in Cc , when the main diagonal Cd is left out, is given in Fig. 8 (*b*) as 3,800 pounds, tension. Then, the combined tension in Cc is equal to $3,200 + 3,800 = 7,000$ pounds, which is the minimum stress in Cc .

The maximum tension in Dd occurs when the diagonals Cd and dE are in action, and the truss is loaded as fully as possible without bringing the main diagonal De into action. The maximum live-load tension in dE occurs when the joints b , c , and d are loaded. It was shown in the preceding paragraphs that under this loading the main diagonal dC is in action. The stress in dE due to loads on the left is evidently the same as the stress in $d'E$ due to loads on the right. In Fig. 9 (*d*), the stress in $d'E$ is represented by the vector 17-18, which is 14,800 pounds, tension; the left reaction for loads at b' , c' , and d' is equal to

$$\frac{15,750 \times (1 + 2 + 3)}{8} = 11,800 \text{ pounds}$$

and the actual live-load stress is

$$\frac{11,800}{14,800} \times 14,800 = -17,500 \text{ pounds}$$

From Fig. 8 (*b*), the dead-load stress in dE when the main diagonal De is left out is equal to 1,600 pounds, compression (vector 15'-16'). Then, the combined stress in dE when there are live loads at b , c , and d is equal to $17,500 - 1,600 = 15,900$ pounds, tension.

It remains to be seen how many joints to the right of dE can be loaded without bringing the main diagonal De into action. This can best be determined by considering the vector 15'-16' representing the stress in dE , Fig. 9 (*d*). A left reaction of 10,000 pounds causes a compression in dE equal to 10,600 pounds; then, a reaction of 15,000 pounds will cause a compression in dE equal to 15,900 pounds, which is the numerical value of the combined tension just found. Therefore, if the left reaction due to live loads at e , d' , c' , etc. is less than 15,900 pounds, the counter dE will be in action; if

greater, the main diagonal De will be in action. The left reaction for a live panel load at e is equal to

$$\frac{15,750 \times 4}{8} = 7,900 \text{ pounds}$$

For loads at e and d' , it is

$$\frac{15,750 \times (3 + 4)}{8} = 13,800 \text{ pounds}$$

For loads at e , d' , and c' , it is

$$\frac{15,750 \times (2 + 3 + 4)}{8} = 17,700 \text{ pounds}$$

From which it will be seen that live loads may be placed at the joints e and d' , in addition to those at b , c , and d , without bringing the main diagonal De into action.

The actual live-load tension in Dd may be found by drawing a stress diagram, Fig. 9 (e), of the truss as far as joint D for loads at b , c , d , e , and d' . The vector $14-15'$, equal to 4,200 pounds tension, is the stress in Dd , which may also be found, from Fig. 9 (d), by considering first the tension due to loads at b , c , and d , and then the tension due to loads at e and d' . The former is evidently equal to the vector $18-19'$ (2,100 pounds) multiplied by the ratio of the left reaction due to loads at b' , c' , and d' to 10,000; the latter to vector $14-15'$ (1,200 pounds), multiplied by the ratio of the left reaction due to loads at e and d' to 10,000. They are, respectively, equal to

$$\frac{11,800}{10,000} \times 2,100 = 2,500 \text{ pounds,}$$

$$\text{and} \quad \frac{11,800}{10,000} \times 1,200 = 1,700 \text{ pounds}$$

The dead-load tension in Dd when the main diagonal De is left out is equal to 2,200 pounds [vector $14-15'$, Fig. 8 (b)]. Then, the maximum combined tension in Dd is equal to

$$2,500 + 1,700 + 2,200 = 4,200 + 2,200 = 6,400 \text{ pounds,}$$

which is the minimum stress in Dd .

20. Comparison of Methods.—It will be seen, from the application of the analytic and graphic methods to the preceding example, that the same course of reasoning is required in both, and the same amount of work required in

combining the stresses. The actual determination of the stresses is somewhat shorter by the graphic than by the analytic method, and the results are sufficiently accurate for all practical purposes.

OTHER FORMS OF THE CURVED-CHORD TRUSS

21. The Deck Truss.—The method of calculating the stresses in the members of a deck truss with curved lower chord, such as that represented in Fig. 10, is, in general, the



FIG. 10

same as for the through truss. The principal difference is due to the fact that the minimum stresses in the verticals, as well as the maximum, are compression. The maximum compression in any vertical on the left of the center is equal either to the algebraic sum of the maximum positive shear on the plane of section that intersects such member and two chord members, and the vertical component of the stress in the inclined-chord member for the same loading, or to the sum of a dead and a live panel load, whichever is the greater. It

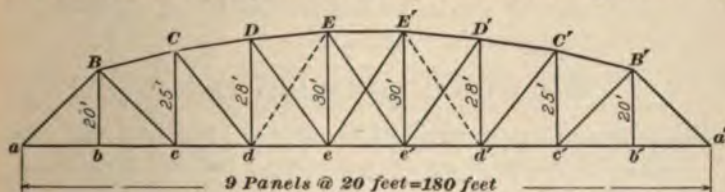


FIG. 11

is evident that the maximum compression in a vertical cannot be less than the panel load supported at its upper joint. In a similar manner, the minimum compression is equal to the algebraic sum of the minimum positive shear on the section and the vertical component of the stress in the

inclined-chord member for the same loading, if the algebraic sum is greater than a dead panel load; if less, the minimum compression is equal to a dead panel load.

22. Odd Number of Panels.—When a curved-chord truss has an odd number of panels, as in Fig. 11, the chords in the center panel are made parallel, and the stresses in the members in that panel are found in the same way as for parallel-chord trusses.

EXAMPLES FOR PRACTICE

1. Fig. 10 represents a deck truss with dimensions as shown. The dead panel load is 7,000, and the live panel load, 15,750 pounds. Assuming that all the dead load is applied at the joints of the loaded chord, find the maximum and minimum combined stresses in bB and cC .

Ans.	MEMBER	STRESS, IN POUNDS	
		MAXIMUM	MINIMUM
	bB	+ 61,900	+ 19,000
	cC	+ 47,500	+ 10,600

2. Fig. 11 represents a curved-chord through truss with dimensions as shown. The dead load is 1,500, and the live load, 2,800, pounds per linear foot of bridge, one-half being carried by each truss. Determine the maximum stresses in the main diagonals and counters in the panels in which counters are required.

Ans.	MEMBER	MAXIMUM STRESS, IN POUNDS
	Dc	- 48,200
	dE	- 25,600
	$Ee', E'e$	- 37,400

THE INCLINED-CHORD TRUSS

23. Description.—Figs. 12 and 13 represent the type of inclined-chord truss most frequently used for bridge purposes. When the truss has an even number of panels, the inclined chord is usually made straight from the center of the truss to the first panel point from the end. When there is an odd number of panels, the center chord member is made horizontal, and the rest of the chord inclined. In some cases, when there is an even number of panels, the two chord members in the panels at the center are made

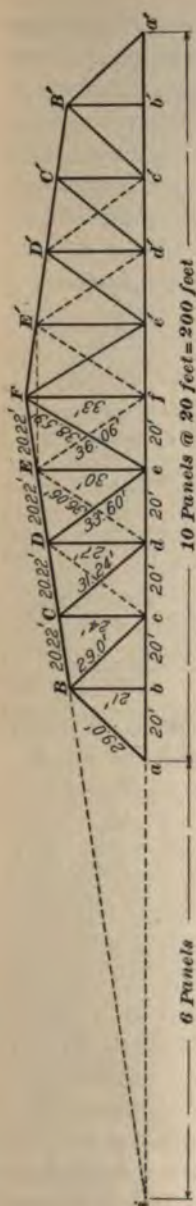


FIG. 12

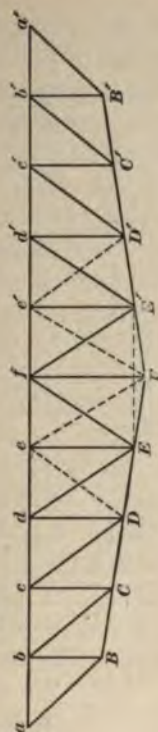


FIG. 13

horizontal, as represented by the dotted lines EE' in Figs. 12 and 13.

24. Method of Calculation.—The general method of analysis is the same as for the curved-chord truss. The actual calculation is somewhat simpler, because the inclined chord has the same inclination in the different panels; there is, therefore, but one intersection of the top and bottom chords to be considered in finding web stresses by the method of moments. In all the trusses analyzed in the preceding pages, the diagonals that slope upwards from the center toward the end have been

called the main diagonals, and have been in tension for a full live load, or for no live load. In the inclined-chord truss, it frequently happens that, under one of these conditions of loading, the vertical component of the stress in an inclined-chord member near the center is numerically greater than the shear in the panel, in which case the diagonal that slopes upwards from the center toward the end is out of action and there is tension in the other diagonal; this will be illustrated presently by an example. In such a truss, the diagonal that slopes downwards from

the center toward the end in that panel is called the main diagonal, and the other the counter. It is necessary to find which diagonal is in action in each panel before the chord stresses can be found; this can be determined very readily by assuming that the stress in either diagonal is tension and writing the expression for the value of the vertical component of the stress in that diagonal, taking moments about the intersection of the two chord members produced. If the vertical component comes out positive, the assumed diagonal is in action; if negative, the other diagonal is in action. As in the curved-chord truss, the panels in which counters are required cannot be determined directly from the shears. If the maximum and minimum combined stresses in any diagonal are found to be tension, no counter is required in that panel; if the maximum is found to be tension and the minimum compression, a counter is required. It is only necessary to calculate the minimum stresses in the diagonals from the end up to the first panel in which a counter is required; a counter is required in all panels from this up to the center and the minimum stresses in the diagonals in these panels are all zero. The verticals require special consideration.

25. Through Truss.—Fig. 12 represents a through truss. The stress in the hip vertical is evidently tension and is equal to the load at its lower joint; the maximum tension is equal to the sum of a dead and a live panel load; the minimum tension is equal to a dead panel load. The maximum stress in the other verticals is compression and is found, in the same way as for the curved-chord truss, by loading all joints to the right of the vertical under consideration. The minimum stress in the center vertical is tension; if the diagonals that meet at its lower joint are in action for full load, the maximum tension is equal to the sum of the vertical components of the stresses in the chord members that meet at its upper joint; if the diagonals that meet at its upper joint are in action for full load, the maximum tension is equal to the sum of the dead and the live panel load at its lower joint. The minimum stress in any

other vertical that has a counter adjacent, obtains when the two diagonals that meet at its lower joint are in action. Then the vertical components of the stresses in the chord members that meet at its upper joint are equal and opposite, and the stress in the vertical is equal to zero, or is compression and equal to the dead load at its upper joint, if any. The minimum stress in any vertical with no counter adjacent may be found by loading all joints to the left of the member, including the joint at its lower end.

26. Deck Truss.—Fig. 13 represents a deck truss. The maximum stress in any vertical is compression, and is found, in the same way as for the curved-chord truss, by loading all the joints to the right of the member, including the joint at its upper end; the maximum compression, however, cannot be less than the sum of a dead and a live panel load. The minimum stress is also compression, and is found by loading all joints to the left of the member; the minimum compression cannot be less than the dead panel load at the upper end of the member.

As the analysis of the inclined-chord truss differs in a slight degree from the curved, chord truss, the following example is given as an illustration.

ILLUSTRATIVE EXAMPLE

27. Data.—Fig. 12 represents a ten-panel through inclined-chord truss with dimensions as shown. The dead load is 1,500 pounds per linear foot of bridge; one-third of it is assumed to be applied at the joints of the unloaded chord. The live load is 2,800 pounds per linear foot of bridge. The lengths, in feet, of all the members are shown in the figure. In what follows, the maximum and minimum stresses in all the members will be calculated by the analytic method.

28. Panel Loads and Reactions.—The dead panel load for one truss is

$$\frac{1,500 \times 20}{2} = 15,000 \text{ pounds}$$

of which 5,000 is applied at each of the upper-chord joints, and 10,000 at each of the lower-chord joints. Each dead-load reaction is equal to

$$\frac{15,000 \times 9}{2} = 67,500 \text{ pounds}$$

The live panel load for one truss is

$$\frac{2,800 \times 20}{2} = 28,000 \text{ pounds}$$

and each reaction for full load is equal to

$$\frac{28,000 \times 9}{2} = 126,000 \text{ pounds}$$

29. Counters and Main Diagonals.—To find which diagonals are counters, and which are main diagonals in the panels near the center, the expression for the vertical component of the stress in either diagonal for dead load alone will be written, assuming that the stress is tension; if the result is positive, the diagonal for which the vertical component was found is the main diagonal, and the other the counter; if negative, then the diagonal considered is the counter. The intersection of the chords is six panel lengths to the left of *a*.

Panel ef.—The member *Ef* will be assumed in tension. Then, the vertical component in *Ef* is

$$\frac{67,500 \times 6 - 15,000 \times (7 + 8 + 9 + 10)}{11} = - \frac{105,000}{11}$$

As this comes out negative, *Ef* is the counter and *eF* the main diagonal.

Panel de.—The member *De* will be assumed in tension. Then the vertical component in *De* is

$$\frac{67,500 \times 6 - 15,000 \times (7 + 8 + 9)}{10} = + \frac{45,000}{10}$$

As this comes out positive, *De* is the main diagonal and *dE* the counter, if one is required in this panel. It is evident that *Cd* and *Bc* are main diagonals.

30. Dead-Load Chord Stresses.—The dead-load chord stresses are found as follows, the horizontal components in the inclined members being found first:

MEMBER	HORIZONTAL COMPONENT, IN POUNDS
BC	$\frac{(67,500 \times 2 - 15,000 \times 1) \times 20}{24} = 100,000$
CD	$\frac{[67,500 \times 3 - 15,000 \times (2 + 1)] \times 20}{27} = 116,700$
DE, EF	$\frac{[67,500 \times 4 - 15,000 \times (3 + 2 + 1)] \times 20}{30} = 120,000$

From the figure, $\sec H = \frac{20.22}{20}$

Therefore, the dead-load chord stresses, computed from the horizontal components just found, are as follows:

MEMBER	STRESS, IN POUNDS
BC	$100,000 \times \frac{20.22}{20} = + 101,100$
CD	$116,700 \times \frac{20.22}{20} = + 118,000$
DE, EF	$120,000 \times \frac{20.22}{20} = + 121,300$
ab, bc	$\frac{67,500 \times 20}{21} = - 64,300$
cd	$- 100,000 = \text{horizontal component in } BC$
de	$- 116,700 = \text{horizontal component in } CD$
ef	$\frac{[67,500 \times 5 - 15,000 \times (4 + 3 + 2 + 1)] \times 20}{33}$
	$= - 113,600$

31. Live-Load Chord Stresses.—The live-load chord stresses may be conveniently found from the dead-load stresses by multiplying the latter by the ratio of live to dead load, which in this case is $\frac{2,800}{15,000}$, or $\frac{28}{150}$.

32. Maximum and Minimum Chord Stresses.—The minimum stresses in the chords are equal to the dead-load stresses just given. The maximum stresses are equal to the sum of the dead- and live-load stresses, and will be found by multiplying the former by $\frac{2,800 + 1,500}{1,500} = \frac{43}{15}$. They are as follows:

MEMBER	MAXIMUM STRESS, IN POUNDS
<i>BC</i>	$101,100 \times \frac{11}{12} = + 289,800$
<i>CD</i>	$118,000 \times \frac{11}{12} = + 338,300$
<i>DE, EF</i>	$121,300 \times \frac{11}{12} = + 347,700$
<i>ab, bc</i>	$64,300 \times \frac{11}{12} = - 184,300$
<i>cd</i>	$100,000 \times \frac{11}{12} = - 286,700$
<i>de</i>	$116,700 \times \frac{11}{12} = - 334,500$
<i>ef</i>	$113,600 \times \frac{11}{12} = - 325,700$

33. Dead-Load Web Stresses.—The dead-load web stresses are found from the shears and the vertical components of the chord stresses. The shears are:

PANEL	SHEAR, IN POUNDS
<i>ab</i>	67,500
<i>bc</i>	52,500
<i>cd</i>	37,500
<i>de</i>	22,500
<i>ef</i>	7,500

The vertical components of the stresses in the chord members, found by multiplying the horizontal components by $\frac{3}{20}$, which is the tangent of the angle that the upper chord makes with the horizontal, are:

MEMBER	VERTICAL COMPONENTS, IN POUNDS
<i>BC</i>	$100,000 \times \frac{3}{20} = 15,000$
<i>CD</i>	$116,700 \times \frac{3}{20} = 17,500$
<i>DE, EF</i>	$120,000 \times \frac{3}{20} = 18,000$

Now,

$$\begin{aligned}
 \csc B a b &= \frac{29.0}{21} \\
 \csc B c b &= \frac{29.0}{21} \\
 \csc C d c &= \frac{31.24}{24} \\
 \csc D e d &= \frac{33.60}{27} \\
 \csc F e f &= \frac{38.59}{33}
 \end{aligned}$$

The dead-load stresses in the web members, when the main diagonals are in action, can now be computed. They are as follows:

MEMBER	STRESS, IN POUNDS
aB	$67,500 \times \frac{29}{21} = + 93,200$
Bb	$- 10,000$
Bc	$(52,500 - 15,000) \times \frac{29}{21} = - 51,800$
Cc	$42,500 - 15,000 = + 27,500$
Cd	$(37,500 - 17,500) \times \frac{31.24}{24} = - 26,000$
Dd	$27,500 - 17,500 = + 10,000$
De	$(22,500 - 18,000) \times \frac{33.60}{27} = - 5,600$
Ee	$+ 5,000$
eF	$(18,000 - 7,500) \times \frac{38.59}{33} = - 12,300$
Ff	$- 10,000$

It will be assumed that the counters cD , dE , and Ef are required. Then, the dead-load stresses in these counters and the adjacent verticals, when the main diagonals Cd , De , and eF are left out, are as follows:

MEMBER	STRESS, IN POUNDS
Ee	$12,500 - 18,000 = - 5,500$
Cc, Dd	$+ 5,000$
cD	$(37,500 - 15,000) \times \frac{33.60}{27} = + 28,000$
dE	$(22,500 - 17,500) \times \frac{36.06}{30} = + 6,000$
Ef	$(17,000 - 7,500) \times \frac{36.06}{30} = + 11,400$
Ff	$17,000 + 2,500 = - 19,500$

34. Live-Load Web Stresses.—The maximum live-load web stresses are obtained most readily by first finding

the vertical components by the method of moments, as explained in Art. 3, and then multiplying each vertical component by $\csc H$, if the member is inclined.* The stresses, in pounds, are as follows:

Maximum live-load stress in aB ,

$$126,000 \times \frac{2.9}{21} = +174,000$$

Maximum live-load stress in Bb ,

$$-28,000$$

Maximum live-load stress in Bc ,

$$\left[\frac{28,000 \times (1 + 2 + 3 + 4 + 5 + 6 + 7 + 8)}{10} \times \frac{8}{8} \right] \times \frac{2.2}{21} \\ = -104,400$$

Maximum live-load stress in Cc ,

$$\frac{28,000 \times (1 + 2 + 3 + 4 + 5 + 6 + 7)}{10} \times \frac{8}{8} = +58,800$$

Maximum live-load stress in Cd ,

$$\left[\frac{28,000 \times (1 + 2 + 3 + 4 + 5 + 6 + 7)}{10} \times \frac{8}{9} \right] \times \frac{31.24}{24} \\ = -68,000$$

Maximum live-load stress in Dd ,

$$\frac{28,000 \times (1 + 2 + 3 + 4 + 5 + 6)}{10} \times \frac{8}{9} = +39,200$$

Maximum live-load stress in De ,

$$\left[\frac{28,000 \times (1 + 2 + 3 + 4 + 5 + 6)}{10} \times \frac{8}{10} \right] \times \frac{33.60}{27} \\ = -43,900$$

Maximum live-load stress in Ee ,

$$\frac{28,000 \times (1 + 2 + 3 + 4 + 5)}{10} \times \frac{8}{10} = +25,200$$

Maximum live-load stress in Ef ,

$$\left[\frac{28,000 \times (1 + 2 + 3 + 4 + 5)}{10} \times \frac{8}{11} \right] \times \frac{36.06}{30} = -27,500$$

Maximum live-load stress in Ff ,

$$\frac{28,000 \times (1 + 2 + 3 + 4)}{10} \times \frac{1}{11} = + 15,300$$

Maximum live-load stress in $F'e' (Fe)$,

$$\left[\frac{28,000 \times (1 + 2 + 3 + 4)}{10} \times \frac{1}{10} \right] \times \frac{38.59}{33} = - 52,400$$

Maximum live-load stress in $E'e'$,

$$\frac{28,000 \times (1 + 2 + 3)}{10} \times \frac{1}{8} = + 26,900$$

Maximum live-load stress in $E'd' (Ed)$,

$$\left[\frac{28,000 \times (1 + 2 + 3)}{10} \times \frac{1}{9} \right] \times \frac{36.06}{30} = - 35,900$$

Maximum live-load stress in $D'e' (Dc)$,

$$\left[\frac{28,000 \times (1 + 2)}{10} \times \frac{1}{8} \right] \times \frac{33.60}{27} = - 20,900$$

In case counters are not required in the panels bc , cd , and de , it is necessary to know the minimum live-load stresses in the members adjacent to those panels. For this reason, they will be calculated now. They are as follows:

MEMBERS	MINIMUM LIVE-LOAD STRESS, IN POUNDS
Bb	0
Bc	$\left[\frac{28,000 \times 1}{10} \times \frac{1}{8} \right] \times \frac{22}{11} = + 7,700$
Cc	$\frac{28,000 \times (1 + 2)}{10} \times \frac{1}{8} = - 16,800$
Cd	$\left[\frac{28,000 \times (1 + 2)}{10} \times \frac{1}{9} \right] \times \frac{31.24}{24} = + 19,400$
Dd	$\frac{28,000 \times (1 + 2 + 3)}{10} \times \frac{1}{8} = - 29,900$
De	$\left[\frac{28,000 \times (1 + 2 + 3)}{10} \times \frac{1}{10} \right] \times \frac{33.60}{27} = + 33,400$

35. Maximum and Minimum Combined Web Stresses.—The maximum and minimum web stresses due to combined live and dead load are as follows:

Member	Maximum Live-Load Stress	Minimum Live-Load Stress	Dead-Load Stress	Dead-Load Counter Stress	Maximum Combined Stress	Minimum Combined Stress
<i>aB</i>	+ 174,000		+ 93,200		+ 267,200	+ 93,200
<i>Bc</i>	- 104,400	+ 7,700	- 51,800		- 156,200	- 44,100
<i>Cd</i>	- 68,000	+ 19,400	- 26,000		- 94,000	- 6,600
<i>De</i>	- 43,900	+ 33,400	- 5,600		- 49,500	+ 27,800
<i>Ef</i>	- 27,500			+ 11,400	- 16,100	
<i>Fe</i>	- 52,400		- 12,300		- 64,700	
<i>Ed</i>	- 35,900			+ 6,000	- 29,900	
<i>Dc</i>	- 20,900			+ 28,000	+ 7,100	
<i>Bb</i>	- 28,000		- 10,000		- 38,000	- 10,000
<i>Cc</i>	+ 58,800	- 16,800	+ 27,500		+ 86,300	+ 10,700
<i>Dd</i>	+ 39,200		+ 10,000	+ 5,000	+ 49,200	+ 5,000
<i>Ee</i>	+ 25,200		+ 5,000	- 5,500	+ 19,700	+ 5,000
<i>E'e'</i>	+ 26,900			+ 500	+ 27,400	
<i>Ff</i>	- 28,000	+ 15,300	- 10,000	- 19,500	- 38,000	- 4,200

It is found that the stress in *Ff* due to a full live load is numerically greater than that previously found for partial load.

Since a counter is required in panel *dc*, there will be no live-load tension in the vertical *Dd*; for, when the joints to the left of *d* are loaded, the counter *dE* will be in action, and the stress in *Dd* is then the dead load of 5,000 pounds at *D*. The stress in *E'e'* for loads at *d'*, *c'*, and *b'* is found to be greater than that in *Ef* for loads from *f* to *b'*.

EXAMPLES FOR PRACTICE

1. In Fig. 14 is represented an eleven-panel inclined-chord truss

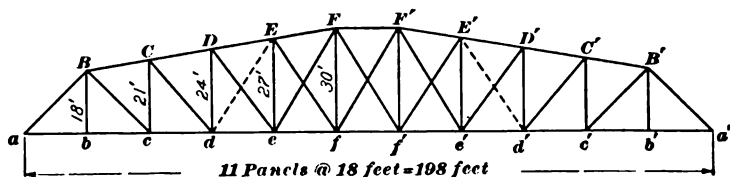


FIG. 14

with dimensions as shown. The dead load is 1,500, and the live load,

2,800 pounds per linear foot of bridge, one-half of the load being carried by one truss. Assuming that one-third of the dead load is applied at the joints of the unloaded chord, find the maximum combined stresses in the diagonals De and dE .

Ans.	MEMBER STRESS, IN POUNDS	
	De	- 52,000
	dE	- 24,900

2. Referring to the same truss as in the preceding example, find the maximum combined stress in the chord member DE .

Ans. + 366,200 lb.

3. For the truss referred to in the preceding example, determine the maximum and minimum combined stresses in Ee .

Ans.	STRESS, IN POUNDS	
	MAXIMUM	MINIMUM
	+ 23,700	+ 4,500

THE PETIT TRUSS

36. Description.—When the method of subdivision used in the Baltimore truss (*Stresses in Bridge Trusses*, Part 2) is applied to the simple type of curved-chord truss, as shown in Figs. 15 and 16, the truss is called a *Petit truss*. This form of truss is well adapted to long spans, being very economical, and is at present the standard type of bridge truss in America for the longest spans in which simple trusses are

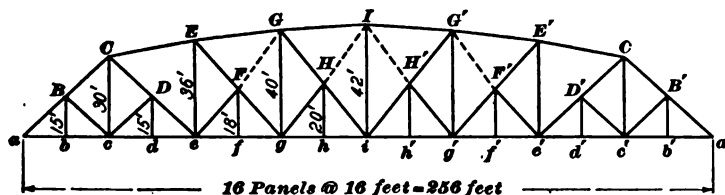


FIG. 15

used. One of the longest simple-truss spans ever built is composed of Petit trusses 675 feet center to center of end supports, and 130 feet center to center of chords at the center of the truss. For very long spans, such as this, the modern tendency is toward the use of very long panels, those in the bridge mentioned being greater than 50 feet. Either substruts, as in Fig. 15, or subties, as in Fig. 16,

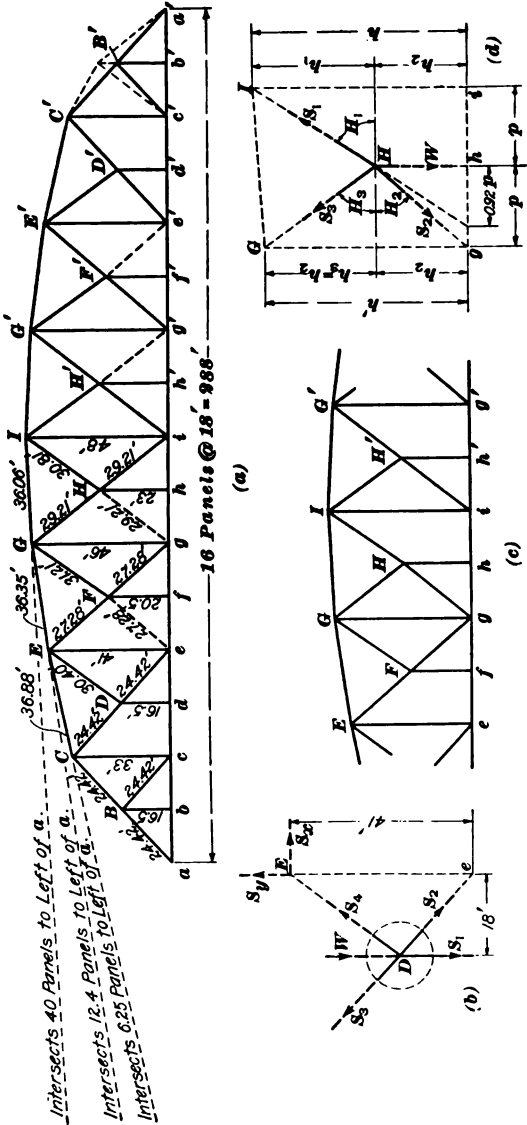


FIG. 16

may be used; the latter type is used when it is desired to make all the diagonals tension members. In both types, the short verticals divide the loaded chord (in this case the lower chord) and the main diagonal into two equal parts; the short diagonals are run from the intersection of the short vertical and main diagonal to the opposite chord, and therefore are not in straight lines with the respective counters.

37. Method of Calculation.—The method of analysis is precisely the same as for the Baltimore and the curved-chord truss, and the student should be thoroughly familiar with these trusses before attempting the analysis of the Petit truss. It will be impossible, however, for him to make a mistake if he applies the conditions of equilibrium correctly. In case of doubt regarding the stress in any member, the following steps should be taken in order:

1. *Consider the loading that produces the desired stress.*
2. *Determine which members are in action for that loading.*
3. *Consider the truss cut into two parts by a surface that intersects the member in which the stress is desired, and not more than two other members in which the stresses are unknown.*
4. *Consider the part of the truss on one side of the cutting surface, drawing a figure representing this part if necessary, replacing by external forces the stresses in the members cut.*
5. *Apply such equation or equations of equilibrium to the part considered as will give the desired stress in the most convenient way.*

To illustrate the method of calculation, the stresses will be found in the form of truss having subties, as represented in Fig. 16. This is a sixteen-panel through Petit truss with dimensions as shown. The dead load will be taken as 2,400, and the live load as 3,000, pounds per linear foot of bridge, one-half being carried by one truss. One-third of the dead load will be assumed to be applied at the upper and intermediate joints. The lengths of all the inclined members are given in the figure.

38. Panel Loads and Reactions.—The dead panel load for each truss is

$$\frac{2,400 \times 18}{2} = 21,600 \text{ pounds}$$

of which 14,400 pounds is applied at each of the lower-chord joints, and 7,200 pounds at each of the upper and intermediate joints *B, C, D, E*, etc. Each dead-load reaction is equal to

$$\frac{21,600 \times 15}{2} = 162,000 \text{ pounds}$$

The live panel load for each truss is

$$\frac{3,000 \times 18}{2} = 27,000 \text{ pounds}$$

and each live-load reaction for full load is

$$\frac{27,000 \times 15}{2} = 202,500 \text{ pounds}$$

ANALYTIC METHOD

39. Chord Stresses.—In the panels *cf, fg, gh*, and *hi*, the main diagonals *EFg* and *GHi* and the short diagonals *FG* and *HI* are in action when there is no live load, and when there is a full live load on the truss. The dead-load chord stresses, in pounds, are as follows:

Stress in *ab* and *bc*,

$$\frac{162,000 \times 18}{16.5} = -176,700$$

Stress in *cd* and *dc*,

$$\frac{(162,000 \times 2 - 21,600 \times 1) \times 18}{33} = -164,900$$

Stress in *ef* and *fg*,

$$\frac{[162,000 \times 4 - 21,600 \times (3 + 2 + 1)] \times 18}{41} = -227,600$$

Stress in *gh* and *hi*,

$$\frac{[162,000 \times 6 - 21,600 \times (5 + 4 + 3 + 2 + 1)] \times 18}{46} = -253,600$$

Stress in CE ,

$$\frac{[162,000 \times 4 - 21,600 \times (3 + 2)] \times 18}{41} \times \frac{36.88}{36}$$

$$= 237,100 \times \frac{36.88}{36} = +242,900$$

Stress in EG ,

$$\frac{[162,000 \times 6 - 21,600 \times (5 + 4 + 3 + 2)] \times 18}{46} \times \frac{36.35}{36}$$

$$= 262,000 \times \frac{36.35}{36} = +264,500$$

Stress in GI ,

$$\frac{[162,000 \times 8 - 21,600 \times (7 + 6 + 5 + 4 + 3 + 2)] \times 18}{48}$$

$$\times \frac{36.06}{36} = 267,300 \times \frac{36.06}{36} = +267,700$$

The live-load chord stresses may be found by multiplying the foregoing stresses by $\frac{3,000}{24,000}$, or $\frac{5}{4}$.

40. Maximum and Minimum Chord Stresses.

The dead-load stresses just given are the minimum stresses. The maximum combined chord stresses will be found by multiplying the minimum stresses by $\frac{3,000 + 2,400}{24} = \frac{9}{4}$.

They are as follows:

MEMBER	MAXIMUM STRESS, IN POUNDS
ab, bc	$176,700 \times \frac{9}{4} = -397,600$
cd, de	$164,900 \times \frac{9}{4} = -371,000$
ef, fg	$227,600 \times \frac{9}{4} = -512,100$
gh, hi	$253,600 \times \frac{9}{4} = -570,600$
CE	$242,900 \times \frac{9}{4} = +546,500$
EG	$264,500 \times \frac{9}{4} = +595,100$
GI	$267,700 \times \frac{9}{4} = +602,300$

41. Dead-Load Web Stresses.—The dead-load stress in each of the subverticals Bb , Dd , Ff , and Hh is $-14,400$ pounds.

1. *Short Diagonals.*—As the short diagonal in any panel does not make the same angle with the horizontal as the main diagonal to which it connects, its stress will not be equal to the product of one-half the load at an intermediate

joint and the cosecant of the angle H , as in the case of the Baltimore truss. This stress is found most easily by considering one of the intermediate joints, such as D . The forces that act at joint D are shown in Fig. 16 (b): S_1 , the stress in the short diagonal, may be replaced at E in its line of action by its vertical and horizontal components S_v and S_x , respectively. If the center of moments is taken at c , the moments of S_1 , S_v , and S_x will all be zero; and the equation of moments is:

$$\Sigma M = (S_1 + W) 18 - S_x \times 41 = 0;$$

whence

$$S_x = \frac{(S_1 + W) \times 18}{41}$$

Then,

$$S_1 = \frac{(S_1 + W) \times 18}{41} \times \frac{30.40}{18} = \frac{(S_1 + W) \times 30.40}{41}$$

$$\text{and } S_v = \frac{(S_1 + W) \times 18}{41} \times \frac{24.5}{18} = \frac{(S_1 + W) \times 24.5}{41}$$

From these equations, the following principle may be stated:

The dead-load stress in a short diagonal of the Petit truss with subties is equal to the sum of the stress in a short vertical and the dead load at an intermediate joint, multiplied by the ratio of the length of the short diagonal to the height of the truss at the point where the short diagonal joins the chord; also, the vertical component of the stress in a short diagonal is equal to the sum of the stress in a short vertical and the dead load at an intermediate joint multiplied by the ratio of the vertical projection of the short diagonal to the height of the truss at the point where the short diagonal joins the chord.

The dead-load stresses in the short diagonals are as follows:

MEMBER	DEAD-LOAD STRESS, IN POUNDS
Bc	$(14,400 + 7,200) \times \frac{24.42}{33} = + 16,000$
DE	$(14,400 + 7,200) \times \frac{30.40}{11} = - 16,000$
FG	$(14,400 + 7,200) \times \frac{31.21}{46} = - 14,700$
HI	$(14,400 + 7,200) \times \frac{30.81}{48} = - 13,900$

2. *Main Diagonals*.—The dead-load stress in a main diagonal is found by multiplying the algebraic sum of the shear and the vertical components of the stresses in the other inclined members in the panel in which the member is located, by the cosecant of the angle that the main diagonal makes with the horizontal.

The vertical components of the stresses in the inclined chord members are:

MEMBER	VERTICAL COMPONENT, IN POUNDS
<i>CE</i>	$237,100 \times \frac{4}{38} = 52,700$
<i>EG</i>	$262,000 \times \frac{5}{38} = 36,400$
<i>GI</i>	$267,300 \times \frac{2}{38} = 14,900$

The shears are as follows:

PANEL	SHEAR, IN POUNDS	PANEL	SHEAR, IN POUNDS
<i>ab</i>	162,000	<i>ef</i>	75,600
<i>bc</i>	140,400	<i>fg</i>	54,000
<i>cd</i>	118,800	<i>gh</i>	32,400
<i>de</i>	97,200	<i>hi</i>	10,800

The vertical components of the stresses in the short diagonals are as follows:

MEMBER	VERTICAL COMPONENT OF STRESS, IN POUNDS
<i>Bc</i>	$21,600 \times \frac{16.5}{33} = 10,800$
<i>DE</i>	$21,600 \times \frac{24.5}{41} = 12,900$
<i>FG</i>	$21,600 \times \frac{25.5}{46} = 12,000$
<i>HI</i>	$21,600 \times \frac{25}{18} = 11,300$

The dead-load stresses, in pounds, in the main diagonals are as follows:

Dead-load stress in *aB*,

$$162,000 \times \frac{24.42}{16.5} = 162,000 \times 1.48 = + 239,800$$

Dead-load stress in BC ,

$$(140,400 + 10,800) \times \frac{24.42}{16.5} = 151,200 \times 1.48 = + 223,800$$

Dead-load stress in CD ,

$$(118,800 - 52,700) \times \frac{24.42}{16.5} = 66,100 \times 1.48 = - 97,800$$

Dead-load stress in De ,

$$(97,200 - 52,700 + 12,900) \times \frac{24.42}{16.5} = 57,400 \times 1.48 \\ = - 85,000$$

Dead-load stress in EF ,

$$(75,600 - 36,400) \times \frac{27.28}{20.5} = 39,200 \times 1.33 = - 52,100$$

Dead-load stress in Fg ,

$$(54,000 - 36,400 + 12,000) \times \frac{27.28}{20.5} = 29,600 \times 1.33 \\ = - 39,400$$

Dead-load stress in GH ,

$$(32,400 - 14,900) \times \frac{29.21}{23} = 17,500 \times 1.27 = - 22,200$$

Dead-load stress in Hi ,

$$(10,800 - 14,900 + 11,300) \times \frac{29.21}{23} = 7,200 \times 1.27 = - 9,100$$

3. *Long Verticals.*—The stresses in the long verticals are found by considering the lower joint of each vertical as a free body. They are as follows:

MEMBER	DEAD-LOAD STRESS, IN POUNDS
Cc	$10,800 + 14,400 = - 25,200$
Ee	$57,400 - 14,400 = + 43,000$
Gg	$29,600 - 14,400 = + 15,200$
Ii	$7,200 - 14,400 + 7,200 = 0$

The fact that the stress in the middle vertical Ii is equal to zero, when there is no live load on the truss, is not a general property of the Petit truss; it occurs in this case simply because the sum of the vertical components of the stresses in the two diagonals IIi and iH' that meet at the lower joint of the vertical, is equal to the dead-panel load at its lower joint.

42. Live-Load Web Stresses.—1. The maximum live-load stresses in the hip vertical, short verticals, and short diagonals are as follows:

MEMBER	MAXIMUM LIVE-LOAD STRESS, IN POUNDS
<i>Bb, Dd, Ff, Hh</i>	— 27,000
<i>Cc</i>	$27,000 + 13,500 = - 40,500$
<i>Bc</i>	$27,000 \times \frac{24.42}{33} = + 20,000$
<i>DE</i>	$27,000 \times \frac{30.40}{41} = - 20,000$
<i>FG</i>	$27,000 \times \frac{31.21}{46} = - 18,300$
<i>HI</i>	$27,000 \times \frac{30.81}{48} = - 17,300$

2. *Main Diagonals.*—The maximum live-load stresses in *aB*, *CD*, *EF*, and *GH* occur when the live load extends from the right end up to the joints *b*, *d*, *f*, and *h*, respectively. The stress in *aB* may be found directly from the reaction for full load; the stresses in the other members may be found by the method of moments. The upper-chord members *CE*, *EG*, and *GI* produced intersect the lower chord produced at points distant 6.25, 12.4, and 40 panel lengths to the left of *a*, respectively.

In calculating the left reaction due to a partial live load on the truss, it is well to note that the sum of all whole numbers from 1 to *n* is equal to $\frac{n(n+1)}{2}$; for example, if all the joints from *b'* to *h* are loaded, *n* = 9; and the sum $1 + 2 + 3 + 4 + 5 + 6 + 7 + 8 + 9 = \frac{9 \times (9+1)}{2} = 45$. Then, in finding the left reaction for this loading, the expression may be written

$$\frac{27,000 \times \frac{9 \times (9+1)}{2}}{16}$$

instead of $\frac{27,000 \times (1 + 2 + 3 + 4 + 5 + 6 + 7 + 8 + 9)}{16}$

The stresses in aB , CD , EF , and GH are as follows:

MEMBER	MAXIMUM LIVE-LOAD STRESS, IN POUNDS
aB	$202,500 \times \frac{24.42}{16.5} = + 299,700$
CD	$\frac{27,000 \times \frac{13 \times (13 + 1)}{2}}{16} \times \frac{6.25}{10.25} \times \frac{24.42}{16.5} = - 138,600$
EF	$\frac{27,000 \times \frac{11 \times (11 + 1)}{2}}{16} \times \frac{12.4}{18.4} \times \frac{27.28}{20.5} = - 99,900$
GH	$\frac{27,000 \times \frac{9 \times (9 + 1)}{2}}{16} \times \frac{40}{48} \times \frac{29.21}{23} = - 80,400$

The maximum stresses in BC , De , Fg , and Hi occur when the right end of the truss is loaded; the live load will extend at least as far as c , e , g , and i , respectively; whether or not b , d , f , and h must be loaded will be determined by trial. When the correct loading for any member has been found, the stress may be calculated by multiplying $\csc H$ by the vertical component of the stress found by applying the equation $\sum Y = \sum S \sin H = 0$ to all the forces that meet at the intermediate joint at the end of the member, or to the stresses in all the members cut by a vertical plane of section that intersects the member under consideration. The former method is shorter for De , Fg , and Hi ; the latter for BC .

For the member BC , a panel load will be tried at b . This load alone would cause a negative shear in panel bc equal to 1,700 pounds, and compression in Bc , the vertical component of which would be 13,500 pounds. Then, the load at b would cause a stress in BC equal to

$$(13,500 - 1,700) \times \csc H = + 11,800 \csc H$$

As this is compression, the maximum compression will occur when the truss is fully loaded. For this loading, the stress in BC is

$$(175,500 + 13,500) \times \frac{24.42}{16.5} = + 279,700 \text{ pounds}$$

For the member De , a panel load will be tried at d . This load alone would cause tension in CD and DE , the vertical components of which are:

$$\frac{27,000 \times 13}{16} \times \frac{6.25}{10.25} = 13,400 \text{ in } CD$$

and $27,000 \times \frac{24.5}{41} = 16,100 \text{ in } DE$

Applying the equation $\Sigma Y = \Sigma S \sin H = 0$ to the forces acting at the joint D , and multiplying the result by $\csc H$ gives $-2,500 \csc H$ for the stress in De that would be caused by a load at d . As this is tension, the maximum tension in De will occur when the truss is loaded from the right end up to d . For this loading, the vertical component in CD is

$$\frac{27,000 \times \frac{13 \times (13 + 1)}{2}}{16} \times \frac{6.25}{10.25} = 93,600 \text{ pounds}$$

Applying the equation $\Sigma Y = \Sigma S \sin H = 0$ to the forces acting at the joint D , and multiplying the result by $\csc H$, gives, for the maximum stress in De ,

$$(93,600 + 16,100 - 27,000) \times \frac{24.42}{16.5} = -122,400 \text{ pounds}$$

For the member Fg , a panel load will be tried at f . This load alone would cause tension in EF and FG , the vertical components of which are:

$$\frac{27,000 \times 11}{16} \times \frac{12.4}{18.4} = 12,500 \text{ in } EF$$

and $27,000 \times \frac{25.5}{46} = 15,000 \text{ in } FG$

Applying the equation $\Sigma Y = \Sigma S \sin H = 0$ to the forces acting at joint F , and multiplying the result by $\csc H$, gives $-500 \csc H$ for the stress in Fg that would be caused by a load at f . As this is tension, the maximum tension in Fg will occur when the truss is loaded from the

right end up to f . For this loading, the vertical component in EF is

$$\frac{27,000 \times \frac{11 \times (11 + 1)}{2}}{16} \times \frac{12.4}{18.4} = 75,100 \text{ pounds}$$

and the stress in Fg is

$$(75,100 + 15,000 - 27,000) \times \frac{27.28}{20.5} = -84,000 \text{ pounds}$$

For the member Hi , a panel load will be tried at h . This load alone would cause tension in GH and HI , the vertical components of which are

$$\frac{27,000 \times 9}{16} \times \frac{4.9}{4.8} = 12,700 \text{ in } GH$$

and

$$27,000 \times \frac{2.5}{4.8} = 14,100 \text{ in } HI$$

Applying the equation $\sum Y = \sum S \sin H = 0$ to all the forces acting at the joint H , and multiplying the result by $\csc H$, gives $+200 \csc H$ for the stress in Hi that would be caused by a load at h . As this is compression, the maximum tension in Hi will occur when the truss is loaded from the right end up to i . For this loading, the vertical component in GH is

$$\frac{27,000 \times \frac{8 \times (8 + 1)}{2}}{16} \times \frac{4.9}{4.8} = 50,600 \text{ pounds}$$

and the stress in Hi is

$$50,600 \times \frac{29.21}{23} = -64,300 \text{ pounds}$$

3. *Main Verticals.*—The maximum stresses in the verticals Ee , Gg , and Ii occur when the live load extends from the right end up to f , h , and h' , respectively. When the live load extends up to f , there is no load at d and the live-load stress in DE is zero. Then, if the truss is considered cut by a plane that intersects CE , DE , Ee , and ef , there will be but three unknown stresses, and the stress in Ee may be found by taking moments about the intersection of CE and ef . In like manner, the stress in Gg may be found by taking moments about the intersection of EG and gh , and that

in Ii by taking moments about the intersection of GI and iH' . The results are as follows:

MEMBER	MAXIMUM LIVE-LOAD STRESS, IN POUNDS
Ee	$\frac{27,000 \times \frac{11 \times (11 + 1)}{2}}{16} \times \frac{6.25}{10.25} = + 67,900$
Gg	$\frac{27,000 \times \frac{9 \times (9 + 1)}{2}}{16} \times \frac{12.4}{18.4} = + 51,200$
Ii	$\frac{27,000 \times \frac{7 \times (7 + 1)}{2}}{16} \times \frac{48}{18} = + 39,400$

The dead-load stress in the vertical Ii when there is no live load on the truss—that is, when both main diagonals Hi and iH' are in action—was found in 3, Art. 41, to be zero. When the live load extends from h' to b' , the position that causes the greatest live-load compression in Ii , the main diagonal iH' is out of action, and it is necessary to find the dead-load counter stress in Ii . This stress can be most easily found by considering the joint i . The vertical dead-load forces that act at the joint i are now the dead panel load of 14,400 pounds, the vertical component of the dead load in Hi , 7,200 pounds (2, Art. 41), and the stress in Ii ; the latter is 7,200 pounds, tension. Then, the combined stress in Ii when the live load extends from h' to b' is $39,400 - 7,200 = 32,200$ pounds, compression.

43. Counters and Minimum Stresses in Main Diagonals.—To find the panels in which counters are required, the minimum combined stress will be calculated in each main diagonal, starting at the left end of the truss. The minimum live-load stresses in aB and BC are each equal to zero; then, the minimum stresses are the dead-load stresses.

The minimum live-load stresses in CD and De occur when the joints b and c are loaded; it is evident that a load at d will cause tension in CD , and it was shown in Art. 42 that

a load at d causes tension in De ; then, this joint and all to the right of it must be unloaded. As there is no load at d , the live-load stresses in Dd and DE are equal to zero, and the live-load stress in CD is equal to the live-load stress in De . Considering the truss cut by a plane that intersects CE , CD , and cd , and applying the equation $\sum M = 0$ to the forces acting on the part of the truss to the right of this plane, taking moments about the intersection of CE and cd , gives the minimum live-load stress in CD and De as follows:

$$\frac{27,000 \times (1 + 2)}{16} \times \frac{22.25}{10.25} \times \frac{24.42}{16.5} = +16,300 \text{ pounds}$$

The dead-load stresses in CD and De are equal, respectively, to $-97,800$ and $-85,000$ pounds (2, Art. 41). Then, the minimum combined stresses in CD and De are as follows:

MEMBER	MINIMUM COMBINED STRESS, IN POUNDS
CD	$-97,800 + 16,300 = -81,500$
De	$-85,000 + 16,300 = -68,700$

As these are both tension, no counters are required in the panels cd and de .

The minimum live-load stresses in EF and Fg occur when the joints b , c , d , and e are loaded; it is evident that a load at f will cause tension in EF , and it was shown in Art. 42 that a load at f causes tension in Fg ; then, this joint and all to the right of it must be unloaded. As there is no load at f , the live-load stresses in fF and FG are equal to zero, and the live-load stress in EF is equal to the live-load stress in Fg . Considering the truss cut by a plane that intersects EG , EF , and ef , and applying the equation $\sum M = 0$ to the forces acting on the part of the truss to the right of this plane, taking moments about the intersection of EG and ef , gives the minimum live-load stresses in EF and Fg as follows:

$$\frac{27,000 \times (1 + 2 + 3 + 4)}{16} \times \frac{28.4}{18.4} \times \frac{27.28}{20.5} = +34,700 \text{ pounds}$$

The dead-load stresses in EF and Fg are equal, respectively, to $-52,100$ and $-39,400$ pounds (2, Art. 41). Then, the minimum combined stresses in EF and Fg are as follows:

MEMBER	MINIMUM COMBINED STRESS, IN POUNDS
EF	$-52,100 + 34,700 = -17,400$
Fg	$-39,400 + 34,700 = -4,700$

As these are both tension, no counters are required in the panels ef and fg .

The minimum combined stress, $-4,700$ pounds, in Fg , is so small, and the minimum combined stresses in the main diagonals decrease so rapidly as the center is approached, that it is evident that the minimum combined stress in GH will come out compression. Then, gH acts as the lower half of the counter gHI ; GH acts as a short diagonal or subtie; and HI is out of action, as represented in Fig. 16 (c). As the lower and upper halves, gH and HI , of the counter gHI are not in the same straight line, the stress in the subtie cannot be found by the principle explained in Art. 41, but requires separate consideration. Applying the equations $\Sigma X = \Sigma S \cos H = 0$ and $\Sigma Y = \Sigma S \sin H = 0$ to all the forces acting at the joint H , as represented in Fig. 16 (d), letting W represent the sum of the loads at H and h , and assuming that the stresses in gH , GH , and HI are tension, the following equations are obtained:

$$\Sigma Y = S_2 \sin H_2 - S_3 \sin H_3 + S_1 \sin H_1 - W = 0$$

$$\Sigma X = S_2 \cos H_2 + S_3 \cos H_3 - S_1 \cos H_1 = 0$$

Substituting for the trigonometric functions their algebraic values, and making the proper reductions, the following values of the vertical components of S_2 and S_3 , in terms of the vertical components of S_1 and W , are obtained:

$$S_{2y} = S_2 \times \frac{h_2}{l_2} = S_{1y} \times \frac{\frac{h_1 + h_2}{2}}{h_1} - \frac{W}{2} = S_{1y} \times \frac{h}{2h_1} - \frac{W}{2} \quad (1)$$

$$S_{3y} = S_3 \times \frac{h_3}{l_3} = \frac{W}{2} - S_{1y} \times \frac{\frac{h_1 - h_2}{2}}{h_1} = \frac{W}{2} - S_{1y} \times \frac{h - h'}{2h_1} \quad (2)$$

The stress S_i may be found most readily by the method of moments, taking as the center of moments the intersection of GI and hi produced, which is at a distance of 40 panel lengths to the left of a , and considering S_i to be resolved into its vertical and horizontal components at the intersection of its line of action HI with the lower chord, this intersection is at a distance equal to $\frac{33}{40}$, or .92, panel lengths to the left of h , or 46.08 panel lengths to the right of the center of moments. The dead-load counter stresses, in pounds, in gH , GH , and HI are as follows:

Stress in HI ,

$$\frac{162,000 \times 40 - 21,600 \times (41 + 42 + 43 + 44 + 45 + 46 + 47)}{46.08} \times \frac{30.81}{25} = -4,600$$

Stress in GH ,

$$(10,800 + 3,750 \times \frac{2}{50}) \times \frac{29.21}{23} = 13,900$$

Stresses in gH ,

$$(-3,750 \times \frac{48}{50} - 10,800) \times \frac{29.21}{23} = -18,300$$

In writing these equations, it was assumed that the stresses in HI , GH , and gH were tension; the negative results in HI and gH indicate that the dead-load counter stresses in those members are compression.

The maximum live-load stress in HI , when it is a counter, occurs when the live load extends from b to h , and is as follows:

$$\frac{27,000 \times 7 \times (7 + 1)}{16} \times \frac{56}{46.08} \times \frac{30.81}{25} = -70,800 \text{ pounds}$$

For the maximum tension in gH , it is evident that the live load should extend from the left end at least as far as joint g ; whether or not joint h should be loaded will be determined by trial. A live panel load at h alone would cause a tension in HI , the vertical component of which is equal to

$$27,000 \times \frac{7}{16} \times \frac{56}{46.08} = 14,400 \text{ pounds}$$

and a stress in gH , the vertical component of which is found by applying formula 1 and is equal to

$$14,400 \times \frac{4.8}{5.0} - 13,500 = 300 \text{ pounds}$$

As this comes out positive, the stress in gH will be tension, and for the maximum tension in gH the joints b to h should be loaded. For this loading, the live-load stress in gH is equal to

$$(57,400 \times \frac{4.8}{5.0} - 13,500) \times \frac{29.21}{23} = 41,600 \times 1.27 \\ = - 52,800 \text{ pounds}$$

The maximum live-load stress in GH as a main diagonal was found in Art. 42. Now, if the truss is loaded from b to g , the vertical component of the stress in HI is equal to

$$\frac{27,000 \times \frac{6 \times (6+1)}{2}}{16} \times \frac{56}{46.08} = 43,100 \text{ pounds}$$

and the minimum live-load stress in GH is equal to

$$(0 - 43,100 \times \frac{2}{5.0}) \times \frac{29.21}{23} = - 2,200 \text{ pounds}$$

As this comes out negative, the minimum live-load stress in GH is compression.

44. Minimum Stresses in the Verticals.—The minimum live-load stresses in the hip vertical Cc , and in the short verticals Bb , Dd , Ff , and Hh , are equal to zero. The minimum live-load stresses in the main verticals Ee , Gg , and Ii are tension; the loadings that cause the minimum stresses in these members are determined by trial.

Vertical Ee .—It is first necessary to consider the loading that will probably cause the maximum tension in the vertical Ee . In Art. 42, it was found that the maximum compression occurs when all the joints from b' to f are loaded; therefore, for the maximum tension these joints should be unloaded. In finding the stress in Ee due to loads at the left end, the truss may be considered cut by a plane that intersects CE , DE , Ee , and ef . Live panel loads at b and c cause a right reaction equal to $\frac{27,000 \times (1+2)}{16}$. Then,

taking moments, about the intersection of CE and ef , of all the forces acting on the part of the truss to the right of the plane just mentioned (the stress in DE being equal to zero), the stress in Ee due to live loads at b and c only is

$$\frac{27,000 \times (1+2)}{16} \times \frac{22.25}{10.25} = -11,000 \text{ pounds}$$

Now, if any more joints to the left of e are loaded, it will be seen that a panel load at d tends to cause compression, and at e tension, in Ee ; the stress that would be caused by both of these together will be found, as it will be assumed impossible to have a full live load at e and no load at d . It will be convenient to apply the equation $\sum Y = 0$ to all the forces acting on the part of the truss to the right of the plane of section just mentioned.

Thus, the right reaction, acting upwards, is

$$\frac{27,000 \times (3+4)}{16} = 11,800 \text{ pounds}$$

The vertical component in CE , acting upwards, is

$$\frac{27,000 \times (12+13)}{16} \times \frac{4 \times 18}{41} \times \frac{8}{36} = 16,500 \text{ pounds}$$

The vertical component in DE , acting downwards, is

$$27,000 \times \frac{24.5}{41} = 16,100 \text{ pounds}$$

Then, the live-load stress in Ee due to loads at d and e is equal to

$$11,800 + 16,500 - 16,100 = 12,200 \text{ pounds, tension}$$

Adding this value to the tension due to loads at b and c gives 23,200 pounds for the maximum live-load tension in Ee . The dead-load stress in Ee is equal to +43,000 pounds; then, the minimum combined stress is

$$43,000 - 23,200 = +19,800 \text{ pounds}$$

Vertical Gg.—It was shown in Art. 42 that, for the maximum compression in Gg , the joints h to b' must be loaded. Then, for the maximum tension, these joints must be unloaded. There are two conditions when the joints at the left end of the truss are loaded that need to be considered; namely, when GH is in action as a main diagonal,

and when gH is in action as a counter. The vertical component of the dead-load tension in GH as a main diagonal is equal to 17,500 pounds. Live panel loads at b, c, d , etc., decrease the dead-load tension in GH until the live-load compression in GH is greater than the dead-load tension, when gH comes into action as a counter. Denoting the right reaction for live loads at b, c , etc. by R_2'' , and taking moments about the intersection of GI and gh , the vertical component of the live-load compression in GH is found to be $\frac{R_2'' \times 56}{48}$. Placing this equal to 17,500, the vertical component of dead-load tension in GH , and substituting for R_2''

its value $\frac{27,000 \times \frac{n(n+1)}{2}}{16}$, n being the number of joints

loaded at the left end, n is found to be equal to 3.7. This means that, if three joints b, c , and d are loaded, GH will be in action as a main diagonal; if more than three are loaded, gH will be in action as a counter. The live-load stress in Gg due to loads at b, c , and d may be found by taking moments about the intersection of EG and gh , the live-load stress in FG for this loading being equal to zero. Then, the live-load stress in Gg is

$$\frac{27,000 \times (1 + 2 + 3)}{16} \times \frac{28.4}{18.4} = 15,600 \text{ pounds, tension}$$

The dead-load stress in Gg when GH is in action as a main diagonal is equal to 15,200 pounds, compression. Then, the minimum combined stress in Gg for this loading is equal to $15,600 - 15,200 = 400$ pounds, tension.

When the joints b, c, d , and e are loaded with live load, gH is in action as a counter, and the live-load stress in Gg is found by considering the forces that act at joint g . They are the stresses in Fg, Gg , and gH . The vertical component of the stress in Fg is found by taking the moment of the right reaction about the intersection of EG and fg , and is equal to

$$\frac{27,000 \times (1 + 2 + 3 + 4)}{16} \times \frac{28.4}{18.4} = 26,000 \text{ pounds, compression}$$

The vertical component of the live-load stress in gH is obtained from formula 1, Art. 43, first finding the stress in HI for this loading. The vertical component of the stress in gH is

$$\frac{4.8}{5.0} \times \frac{27,000 \times (1 + 2 + 3 + 4)}{16} \times \frac{56}{46.08} \\ = 19,700 \text{ pounds}$$

Then, the live-load stress in Gg is $26,000 - 19,700 = 6,300$ pounds, tension. If the live load is applied to any joints at the right of e , the tension in Gg will decrease.

When the counter gH is in action, the dead-load stress in Gg may be found by considering the forces that act at the joint g . The vertical component of the dead-load stress in Fg is equal to 29,600 pounds (Art. 41), while that in gH is equal to 14,400 pounds (Art. 43). The dead panel load at g is equal to 14,400 pounds. Then, the dead-load stress in Gg is $29,600 - 14,400 - 14,400 = 800$ pounds, compression. The minimum combined stress in Gg for this loading is $-6,300 + 800 = -5,500$ pounds. As this is less than the stress when the joints b , c , and d are loaded, the minimum combined stress in Gg is equal to 5,500 pounds, tension.

Vertical Ii.—The maximum live-load tension in Ii occurs when the truss is fully loaded. It may be found most easily by considering the forces that meet at the joint I . The vertical component of the stress in GI , or $G'I$, may be found from the dead-load stress given in Art. 41, by multiplying the latter by the ratio of the live to the dead load, or $\frac{5}{4}$. The vertical component of the dead-load stress is equal to +14,900 pounds; then, the vertical component of the live-load stress is equal to $14,900 \times \frac{5}{4} = +18,600$ pounds.

The vertical component of the live-load stress in HI is equal to $27,000 \times \frac{2.5}{4.5} = 14,100$ pounds. Then, the stress in Ii is

$$2 \times 18,600 - 2 \times 14,100 = 9,000 \text{ pounds, tension}$$

As the dead-load stress in Ii is equal to zero, the minimum combined stress is also equal to 9,000 pounds, tension.

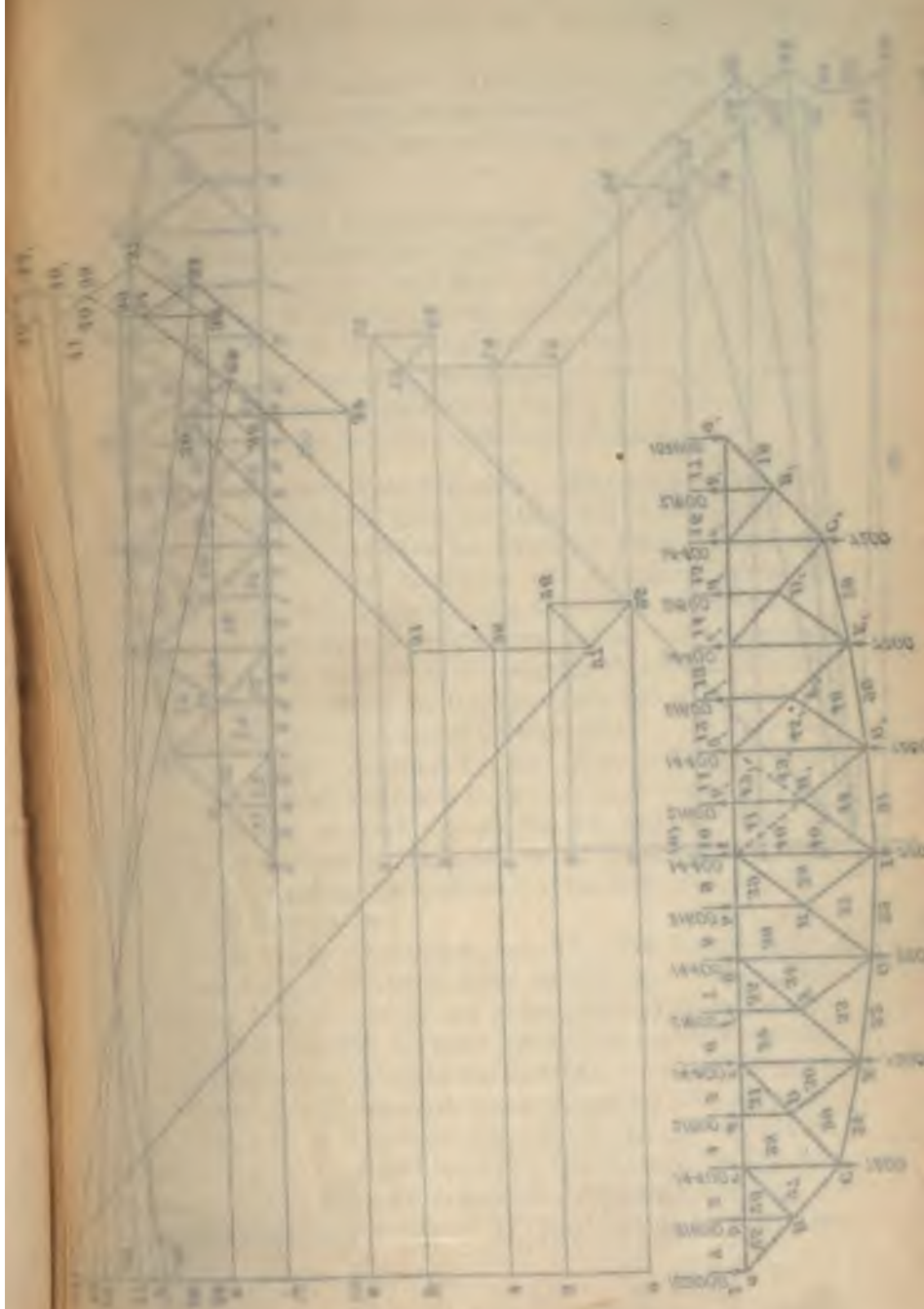
45. Table of Stresses.—The maximum and minimum combined stresses are given in the table on page 63.

Member	Maximum Live-Load Stress	Minimum Live-Load Stress	Dead-Load Stress	Dead-Load Counter Stress	Maximum Combined Stress	Minimum Combined Stress
<i>Bb</i> , etc.	- 27,000		- 14,400		- 41,400	- 14,400
<i>Bc</i>	+ 20,000		+ 16,000		+ 36,000	+ 16,000
<i>DE</i>	- 20,000		- 16,000		- 36,000	- 16,000
<i>FG</i>	- 18,300		- 14,700		- 33,000	- 14,700
<i>HI</i>	- 70,800			+ 4,600	- 66,200	
<i>HI</i>	- 17,300		- 13,900		- 31,200	- 13,900
<i>aB</i>	+ 209,700		+ 239,800		+ 539,500	+ 239,800
<i>BC</i>	+ 279,700		+ 223,800		+ 503,500	+ 223,800
<i>CD</i>	- 138,600	+ 16,300	- 97,800		- 236,400	- 81,500
<i>De</i>	- 122,400	+ 16,300	- 85,000		- 207,400	- 68,700
<i>EF</i>	- 99,900	+ 34,700	- 52,100		- 152,000	- 17,400
<i>Fg</i>	- 84,000	+ 34,700	- 39,400		- 123,400	- 4,700
<i>GH</i>	- 80,400	+ 2,200	- 22,200		- 102,600	
<i>GH</i>				- 13,900		- 11,700
<i>Hi</i>	- 64,300		- 9,100		- 73,400	
<i>gH</i>	- 52,800			+ 18,300	- 34,500	
<i>Ce</i>	- 40,500		- 25,200		- 65,700	- 25,200
<i>Fe</i>	+ 67,900	- 23,200	+ 43,000		+ 110,900	+ 19,800
<i>Gg</i>	+ 51,200	- 6,300	+ 15,200	+ 800	+ 66,400	- 5,500
<i>Ii</i>	+ 39,400	- 9,000		- 7,200	+ 32,200	- 9,000
<i>a b, bc</i>	X 176,700		- 176,700		- 307,600	- 176,700
<i>cd, de</i>	X 164,900		- 164,900		- 371,000	- 164,900
<i>ef, fg</i>	X 227,600		- 227,600		- 512,100	- 227,600
<i>gh, hi</i>	X 253,600		- 253,600		- 570,600	- 253,600
<i>CE</i>	X 242,900		+ 242,900		+ 546,500	+ 242,900
<i>FG</i>	X 264,500		+ 264,500		+ 595,100	+ 264,500
<i>GI</i>	X 267,700		+ 267,700		+ 602,300	+ 267,700

GRAPHIC METHOD

46. Dead-Load Stresses.—The stress diagram for dead-load stresses is represented in Fig. 17 (*b*). In constructing the force polygon (the load line), the work was shortened somewhat by assuming that the dead load that is applied at *B, D, F*, etc. was applied at *b, d, f*, etc., respectively, together with the dead load already there. Then, the external forces acting on the truss are the reactions, equal to 162,000 pounds each, the partial loads at the upper-chord joints, 7,200 pounds each, at *C, E, G*, etc., the full panel loads of 21,600 pounds each, at *b, d, f, h*, etc., and the loads of 14,400 pounds each at *c, e, g, i*, etc., as represented in Fig. 17 (*a*). The result of assuming that the dead loads at *B, D, F*, etc. are carried at *b, d, f*, etc. is that the stress diagram indicates a stress in each short vertical 7,200 pounds greater than the actual stress, and this must be kept in mind when these stresses are scaled from the stress diagram. The stresses in the other members are not affected by the assumption. The points 38 and 40 in the stress diagram coincide, indicating that the stress in *Ii* (38-40) is equal to zero; this checks with the value of the dead-load stress in *Ii*, found in Art. 41 by the analytic method.

In finding the counter stresses in the panels *gh* and *hi*, the stress diagram was drawn for the panels *ih'* and *h'g'* instead; 38-40' is the vector representing the stress in *Ii* when *H'i* is out of action; 40'-44' and 42'-43' are the stresses in *H'I* and *g'H'*, respectively, as counters; 44'-43' is the stress in *H'G'* as a short diagonal, and 43'-45' is the stress in *G'g'*. In drawing the vector 43'-45', the diagram was drawn for joint *G'*. At this joint, 21-44' and 44'-43' were known; 43'-45', 45'-46, and 46-20 were unknown. As there were three unknown forces, it was impossible to draw these three vectors directly. It was known, however, that the stress in *G'E'* is equal to the stress in *GE*, which had already been found; then a line was drawn from 20 parallel to 20-46, and the point 46 located so that 20-46 was equal to 23-33, and, by drawing from 46 a line parallel to 45'-46,



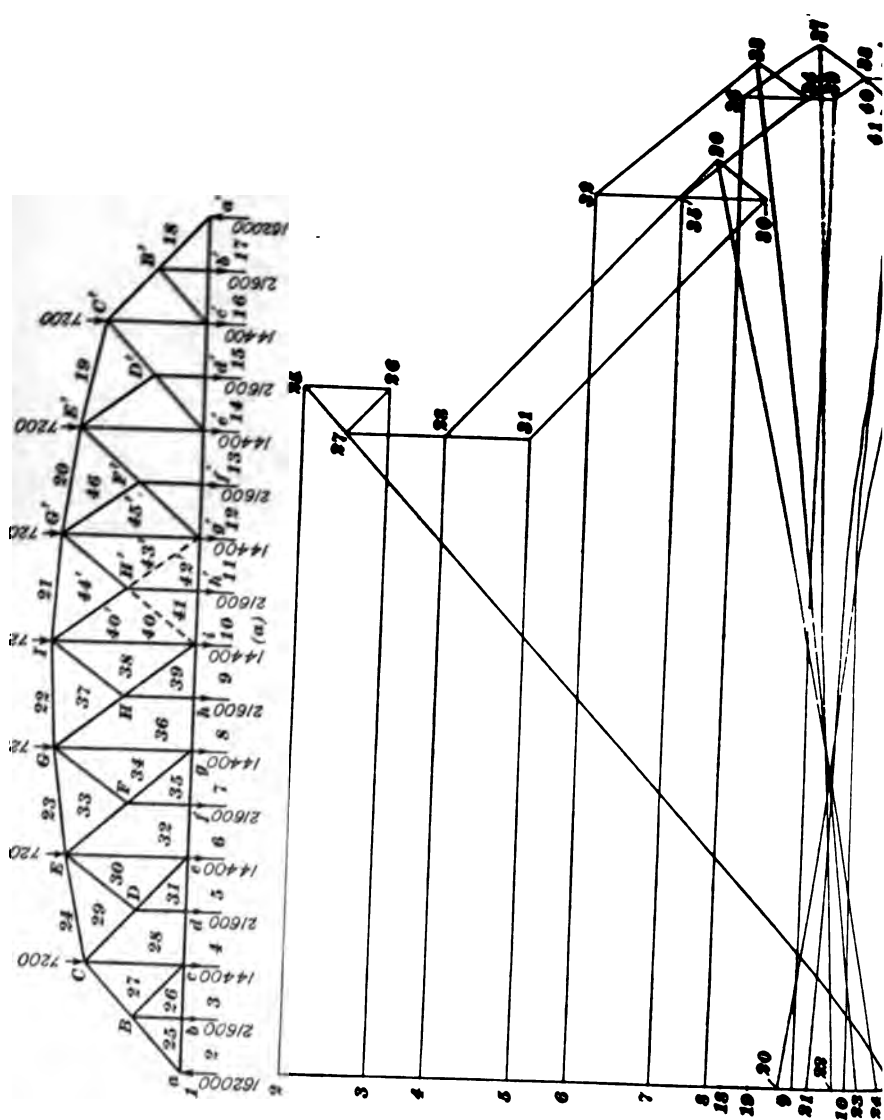
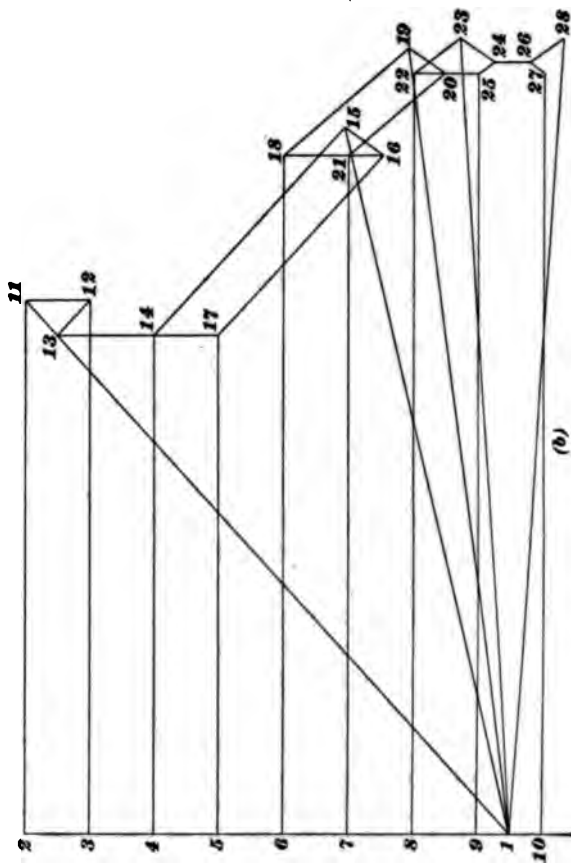
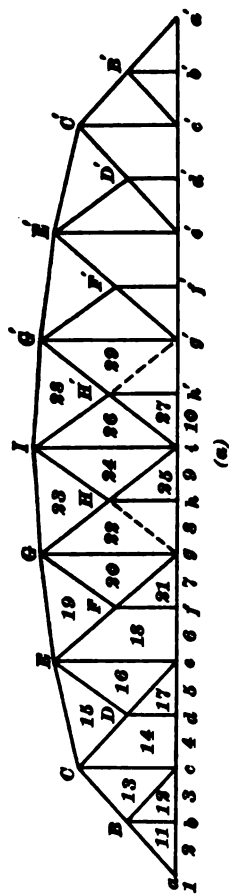


FIG. 17



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and from $43'$ a line parallel to $43'-45'$, the point $45'$ was found. It is unnecessary in the present case to scale the dead-load stresses, since they will be the same as found by the analytic method.

47. Live-Load Chord Stresses.—The chord stresses may be found by drawing a stress diagram for one-half the truss when there is a full live load, as represented in Fig. 18 (*b*), or by multiplying the dead-load chord stresses, scaled from Fig. 17 (*b*), by the ratio of the live to the dead load. In the present case, the former method is preferable, as the stress diagram represented in Fig. 18 (*b*) gives also the desired stresses in some of the web members.

48. Live-Load Web Stresses.—The stresses in the end posts, hip vertical, short verticals, and short diagonals may be scaled directly from the stress diagram represented in Fig. 18 (*b*). The maximum and minimum live-load stresses in the main diagonals and remaining verticals may be found by drawing a stress diagram for the truss, from the left end up to the member $C'e'$, assuming that the only force acting on the left end of the truss is a reaction at a equal to 10,000 pounds, as represented in Fig. 19 (*b*). In this diagram, the vectors representing the stresses in the short verticals and short diagonals do not appear; this is due to the fact that, if no loads act on the left end of the truss except the left reaction, there will be no panel loads at b , d , etc., and consequently the stresses in Bb and Bc , Dd and DE , etc. will be equal to zero.

In finding the stresses in the main diagonals and verticals, it is best to find by the analytic method the loading that causes the desired stress. As these loadings have already been found in Art. 42, the work will not be repeated. When the live-load stress in one of the verticals Ee , Gg , and Ii , or in the upper half of one of the main diagonals CD , EF , and GH , or in IH' as a counter, is a maximum, the only force that acts on the truss to the left of the member is the left reaction. Then, the maximum live-load stress in any one of these members may be found by multiplying the stress in

that member due to a left reaction of 10,000 pounds, as scaled from Fig. 19 (*b*), by the ratio of the left reaction caused by the loading that causes the maximum stress in the member under consideration to the assumed reaction of 10,000 pounds. The stresses in the members just referred to, caused by a left reaction of 10,000 pounds, are as follows:

MEMBER	STRESS, IN POUNDS	VECTOR
<i>Ee</i>	+ 6,100	5'-6
<i>Gg</i>	+ 6,700	7'-8
<i>Ii</i>	+ 8,300	9'-10
<i>CD</i>	- 9,000	4-5
<i>EF</i>	- 9,000	6-7
<i>GH</i>	- 10,600	8-9
<i>IH'</i>	- 15,000	10-11

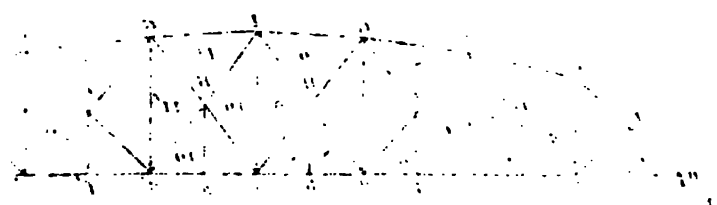
The left reactions caused by the loadings that cause maximum stresses in these members (as found by the analytic method in Art. 42) are as follows:

MEMBER	LEFT REACTION, IN POUNDS	MEMBER	LEFT REACTION, IN POUNDS
<i>Ee</i>	111,400	<i>EF</i>	111,400
<i>Gg</i>	75,900	<i>GH</i>	75,900
<i>Ii</i>	47,300	<i>IH'</i>	47,300
<i>CD</i>	153,600		

Then, the maximum live-load stresses in these members are as follows:

MEMBER	MAXIMUM LIVE-LOAD STRESS, IN POUNDS
<i>Ee</i>	$\frac{111400}{10000} \times 6,100 = + 68,000$
<i>Gg</i>	$\frac{75900}{10000} \times 6,700 = + 50,900$
<i>Ii</i>	$\frac{47300}{10000} \times 8,300 = + 39,300$
<i>CD</i>	$\frac{153600}{10000} \times 9,000 = - 138,200$
<i>EF</i>	$\frac{111400}{10000} \times 9,000 = - 100,300$
<i>GH</i>	$\frac{75900}{10000} \times 10,600 = - 80,500$
<i>IH'</i>	$\frac{47300}{10000} \times 15,000 = - 71,000$

When the stress in the lower half of the main diagonal *Hi* is a maximum, there is no load to the left of *i* except the left reaction, and the stress in *Hi* is equal to the stress



1. The first part of the diagram shows a series of triangles and lines, with points labeled A, B, C, D, E, F, G, H, I, J, K, L, M, N, O, P, Q, R, S, T, U, V, W, X, Y, Z. The lines connect these points to form a series of triangles and other geometric shapes. The diagram is drawn with solid lines for the main structure and dashed lines for auxiliary construction lines. The overall shape is elongated and somewhat irregular, with a series of peaks and valleys. The diagram is likely a technical drawing from a mathematics or engineering textbook, used to illustrate a concept or a proof.

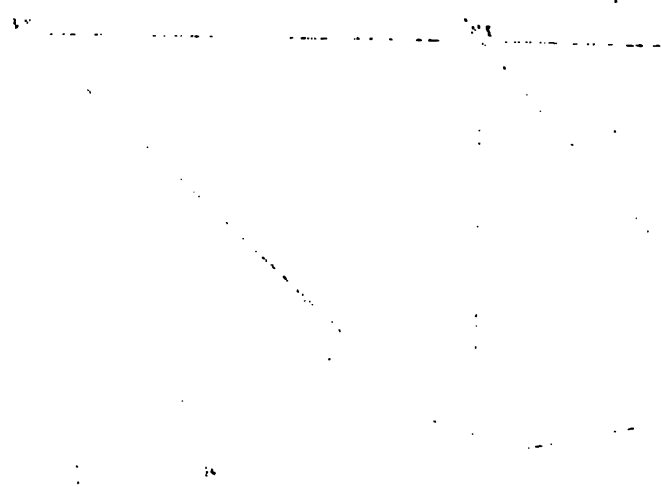
2. The second part of the diagram shows a series of triangles and lines, with points labeled A, B, C, D, E, F, G, H, I, J, K, L, M, N, O, P, Q, R, S, T, U, V, W, X, Y, Z. The lines connect these points to form a series of triangles and other geometric shapes. The diagram is drawn with solid lines for the main structure and dashed lines for auxiliary construction lines. The overall shape is elongated and somewhat irregular, with a series of peaks and valleys. The diagram is likely a technical drawing from a mathematics or engineering textbook, used to illustrate a concept or a proof.

3. The third part of the diagram shows a series of triangles and lines, with points labeled A, B, C, D, E, F, G, H, I, J, K, L, M, N, O, P, Q, R, S, T, U, V, W, X, Y, Z. The lines connect these points to form a series of triangles and other geometric shapes. The diagram is drawn with solid lines for the main structure and dashed lines for auxiliary construction lines. The overall shape is elongated and somewhat irregular, with a series of peaks and valleys. The diagram is likely a technical drawing from a mathematics or engineering textbook, used to illustrate a concept or a proof.

4. The fourth part of the diagram shows a series of triangles and lines, with points labeled A, B, C, D, E, F, G, H, I, J, K, L, M, N, O, P, Q, R, S, T, U, V, W, X, Y, Z. The lines connect these points to form a series of triangles and other geometric shapes. The diagram is drawn with solid lines for the main structure and dashed lines for auxiliary construction lines. The overall shape is elongated and somewhat irregular, with a series of peaks and valleys. The diagram is likely a technical drawing from a mathematics or engineering textbook, used to illustrate a concept or a proof.

5. The fifth part of the diagram shows a series of triangles and lines, with points labeled A, B, C, D, E, F, G, H, I, J, K, L, M, N, O, P, Q, R, S, T, U, V, W, X, Y, Z. The lines connect these points to form a series of triangles and other geometric shapes. The diagram is drawn with solid lines for the main structure and dashed lines for auxiliary construction lines. The overall shape is elongated and somewhat irregular, with a series of peaks and valleys. The diagram is likely a technical drawing from a mathematics or engineering textbook, used to illustrate a concept or a proof.

6. The sixth part of the diagram shows a series of triangles and lines, with points labeled A, B, C, D, E, F, G, H, I, J, K, L, M, N, O, P, Q, R, S, T, U, V, W, X, Y, Z. The lines connect these points to form a series of triangles and other geometric shapes. The diagram is drawn with solid lines for the main structure and dashed lines for auxiliary construction lines. The overall shape is elongated and somewhat irregular, with a series of peaks and valleys. The diagram is likely a technical drawing from a mathematics or engineering textbook, used to illustrate a concept or a proof.



in GH that has just been found. When the stress in the lower half of one of the main diagonals De and Fg , or in $H'g'$ as a counter, is a maximum, the joints d , f , and h' , respectively, at the left of these members are loaded; and, as the left reaction is then not the only force acting on the truss at the left of one of these members, the stress cannot be found by the stress diagram in Fig. 19 (*b*). These are the same loadings, however, that cause the maximum stresses that have just been found in the upper halves of the diagonals, and the stress in the lower half of any one of them may be found by constructing the polygon for the forces that meet at an intermediate joint. For example, the stress in CD , equal to $-138,200$ pounds, is laid off to a convenient scale, as shown in Fig. 19 (*c*) by the line $5-4$, which is parallel to CD ; $4-4'$, equal, by scale, to the stress in Dd (a live panel load), is next drawn parallel to Dd ; then the lines $4'-5'$ and $5-5'$ are drawn parallel, respectively, to De and DE : the line $4'-5'$ represents the stress in De . In like manner, $6'-7'$, Fig. 19 (*d*), represents the stress in Fg , and $10'-11'$, Fig. 19 (*e*), represents the stress in $H'g'$.

The vector $11-11'$, Fig. 19 (*b*), that represents the stress in $H'G'$ when there are no loads to the left of g' except the left reaction of 10,000 pounds, scales $+600$ pounds. The left reaction for the loading that causes the minimum live-load stress in $H'G'$ (Art. 43) is equal to 35,400 pounds. Then, the minimum live-load stress in $H'G'$ as a short diagonal is equal to

$$\frac{35400}{10000} \times 600 = +2100 \text{ pounds}$$

When the stress in one of the main diagonals $g'E'$ or $d'C'$ is a minimum, there is no force acting on the truss to the left of the member under consideration except the left reaction, and the stresses in these members may be found from Fig. 19 (*b*) in the usual way. The stresses due to a left reaction of 10,000 pounds are as follows:

MEMBER	STRESS in pounds	VECTOR
$g'E'$	-2550	$12-12'$
$d'C'$	-3200	$13-13'$

The left reactions that obtain when the stresses in these members have their minimum values are as follows (see Art. 43):

MEMBER	REACTION, IN POUNDS
$g' E'$	16,900
$e' C'$	5,100

Then, the minimum live-load stresses in these members are as follows:

MEMBER	STRESS, IN POUNDS
$g' E'$	$\frac{16,900}{10,000} \times 20,500 = + 34,600$
$e' C'$	$\frac{5,100}{10,000} \times 32,000 = + 16,300$

The minimum stress in Ii occurs when the truss is fully loaded, and may be found from the stress diagram represented in Fig. 18 (*b*). It was found in Art. 44 that the minimum stress in $G'g'$ (Gg) occurs when the joints b', c', d' , and e' are loaded and $g' H'$ is in action as a counter. Then, the minimum stress in $G'g'$ may be found from Fig. 19 (*b*). The vector $12-11'$ scales $- 3,700$ pounds. The left reaction is equal to 16,900 pounds; then, the minimum live-load stress in Gg is equal to

$$\frac{16,900}{10,000} \times 3,700 = - 6,300 \text{ pounds}$$

In Art. 44, it was found that the minimum live-load stress in $E'e'$ occurs when the joints b', c', d' , and e' are loaded. In this case, the stress in $E'e'$ cannot be found from Fig. 19 (*b*), as the vector $12'-13$ was drawn on the assumption that the stress in $E'D'$ is zero; this assumption is not true in the present case, as there is a load at d' . The method of finding the minimum stress in $E'e'$, when the loading is known, is illustrated in Fig. 19 (*f*). The stresses in $G'E'$ and $g'E'$ are found by multiplying the vectors $1-12$ and $12-12'$, respectively, Fig. 19 (*b*), by $\frac{16,900}{10,000}$; the stress in $E'D'$ is the same as the stress in $D'E$, which is given by the vector $5'-5$, Fig. 19 (*c*). Then, as three of the five forces that meet at E' are known, the other two, one of which is the stress in $E'e'$, may be found by constructing the force polygon for the joint E' . The vector $12'-13$, Fig. 19 (*f*), equal to $- 23,200$ pounds, is the minimum live-load stress in $E'e'$.

EXAMPLES FOR PRACTICE

1. The sixteen-panel through Petit truss with subtruss represented in Fig. 15, having dimensions as shown, supports one-half of a bridge. The dead load is 2,200, and the live load, 3,200, pounds per linear foot of bridge. If one-third of the dead load is assumed to be applied at the upper and intermediate joints, what is the maximum combined stress in the member GH ? Ans. - 65,100 lb.

2. For the same truss and loading referred to in example 1, what is the minimum combined stress (maximum tension) in Ii ? Ans. - 60,000 lb.

3. For the same truss and loading referred to in example 1, what is the maximum combined stress in Ee ? Ans. + 81,100 lb.

4. For the same truss and loading referred to in example 1, what is the live-load stress in the member gh ? Ans. - 317,400 lb.

THE MULTIPLE-SYSTEM CURVED-CHORD TRUSS

49. Description.—The truss represented in Fig. 20 (*a*) is known as the **multiple-system curved-chord truss**. It is somewhat similar to the Whipple truss (*Stresses in Bridge Trusses*, Part 2) having two systems of web members, the difference between the two being that the upper chord of one is curved and that of the other is horizontal. The multiple-system curved-chord truss is not in common use at the present time, but, as there are some examples of it in existence, it is well to know how the stresses can be found.

50. Method of Calculation.—If the truss represented in Fig. 20 (*a*) is separated into two systems, as shown in single and double lines in Fig. 20 (*b*) and (*c*), it will be seen that, on account of the angles of the curved chord, each system exerts an upward force on each upper-chord joint of the other system. On this account, the systems do not act independently, and the stresses cannot be found on the assumption that the stresses in each system are caused by the loads that come on that system, unless some other assumption is made.

When there is a full live load or no live load on the truss, the stresses in all the members may be found with a

reasonable degree of accuracy without considering the truss separated into the two systems. If the members GI and $G'I$, Fig. 20 (*b*), were straight, the counters gI and $g'I$ would be out of action when there is a full live load or no live load on the system. In like manner, if FH , HH' , and $H'F'$, Fig. 20 (*c*), were straight, the diagonals fH , Hh' , hH' , and

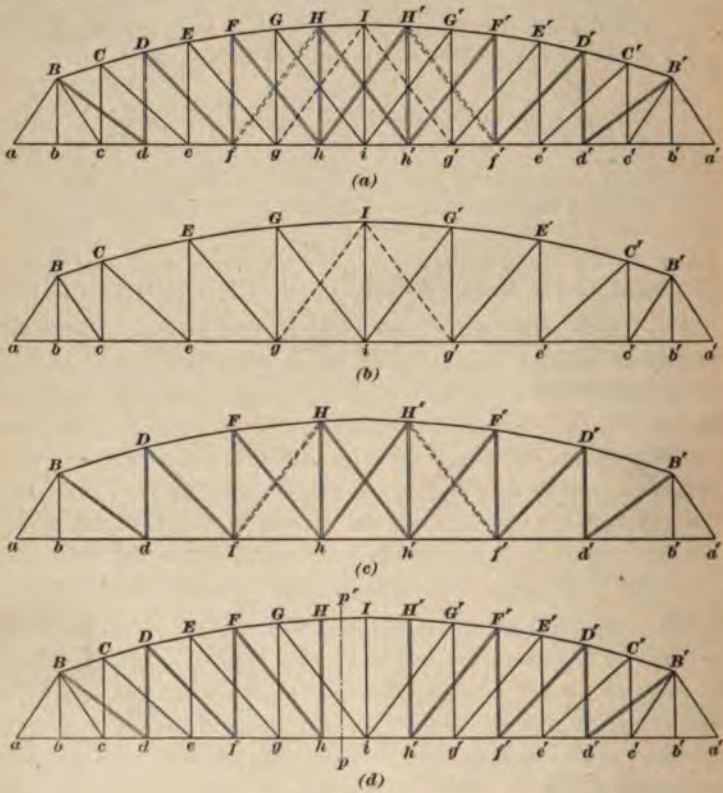


FIG. 20

$H'f'$ would be out of action when there is a full live load or no live load on the system. In the present case, as the upper-chord members referred to are very nearly straight, these diagonals may be assumed to be out of action when there is a full live load or no live load on the truss. Then, the

members of the truss that will be in action for these loadings are shown in Fig. 20 (*d*). If the truss is considered cut by the plane of section $p'p$ that intersects HI , Gi , and hi , and the part to the left of the section is treated as a free body, it will be seen that the stress in HI may be found by taking moments about point i ; the stress in Gi , by applying the equation $\sum Y = \sum S \sin H = 0$ to all the forces acting on the part of the truss considered; and the stress in hi , by applying the equation $\sum X = 0$. The stresses in the remaining members may be found most readily by means of the stress diagram, only one-half of the truss being considered. The stresses in HI , Gi , and hi , considered as external forces, and the panel loads and reaction on the part of the truss to the left of the section $p'p$, form a system in equilibrium, and may be laid off to scale in the force polygon. Then, starting at the right end with the joint H , the stress in HI and the load at H , if any, being known, the stresses in GH and Hh may be found; at joint h , the stresses in Hh and hi and the load at h being known, the stresses in Fh and gh may be found; at joint G , the stresses in GH and Gi and the load at G , if any, being known, the stresses in FG and Gg may be found; etc. By proceeding in this manner, the stresses in all the members, when there is a full live load or no live load on the truss, may be found.

The stresses in the members can also be found by the analytic method of joints; but, as the numerical work in connection with this method when applied to a curved-chord truss is very laborious, its use will not be considered.

51. The maximum live-load stress in almost every web member obtains when there is a partial load on the truss, and cannot be found accurately by the equations of equilibrium unless some assumption is made regarding the upper-chord members. As the upper-chord members of the separate systems, such as EG and GI , Fig. 20 (*b*), are not straight, each system exerts an upward force at each upper-chord joint of the other systems; but, as the actual amount of this force at any joint is relatively small, it may be neglected, and the

stresses found on the assumption that the upper-chord members in the two systems are straight. The stresses in the web members of each system can then be found from the loads that come on that system, in the same way as in the simple type of curved-chord truss explained in Arts. 4 to 20. The graphic method by the stress diagram will give the results with the least work.

OTHER TYPES OF CURVED- AND INCLINED-CHORD TRUSSES

52. Inclined-Chord Truss.—Fig. 21 represents a type of inclined-chord truss in which the joints of the upper chord lie in a straight inclined line from the center to the end of the

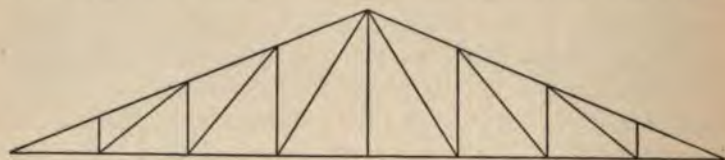


FIG. 21

truss. This type of truss is not now used for bridge purposes, but is employed to a great extent for roofs. It possesses the advantage that the stresses in the web members never reverse; the diagonals are sometimes made tension members and sometimes compression; in the truss shown, they are tension members. When there is a full live load, or no live load on the truss, the stress in every diagonal is tension; when the live load covers a part of the span, the live-load stresses in the diagonals in the unloaded part of the truss are all zero. The method of calculation is precisely the same as for the inclined-chord truss explained in Arts. 23 and following. In finding stresses in the web members by the method of moments, it is well to note that the intersection of the chords, which is the center of moments, is the end joint of the truss, and that the reaction acts at this point. The moment of the reaction about this point is therefore zero, and it is unnecessary to compute the reaction.

53. The Bowstring Truss.—When the joints of the curved chord lie on a curve that passes through the end joints of the horizontal chord, the truss is called a **bowstring truss**. Two common types of this truss are represented in Figs. 22 and 23 as pony trusses. When a bowstring truss is used in



FIG. 22

a deck bridge, the upper chord is horizontal and the lower chord curved. The curved chord is usually made a parabola. The height at the center is sometimes spoken of as the **rise**.

When there is a full live load or no live load on the truss, the stress in the horizontal chord and the horizontal component of the stress in the curved chord are equal and constant from end to end, and the stresses in the diagonals are all zero. The diagonals in Fig. 22 are tension members, and two are required in every panel; one diagonal is in action when one end of the truss is loaded with live load, the other when the other end is loaded. In Fig. 23, the web members are all inclined, and, as there is but one set, they will all get alternate stresses of tension and compression when the truss is partially loaded with live load.

Bowstring trusses are not built at the present time to any great extent, but there are a large number in use. The theo-



FIG. 23

retical advantage in the use of this truss lies in the fact that the maximum chord stresses are almost constant; this is offset to a great extent by the number of counters required, or the reversal of stress in the web members, and the difficulty in getting an efficient system of lateral and sway bracing.

54. The Lenticular Truss.—When both chords are curved, and the joints lie on curves that pass through the end joints of the truss, as represented in Fig. 24, the truss is called a **double bowstring, lenticular, or fish-belly truss**. The curves are usually parabolas; the lengths of the verticals vary as the ordinates to a parabola, and the action of the chords and diagonals is the same as in the bowstring truss represented in Fig. 22.

The floor may be supported along any of the lines ab , cd , or ef , Fig. 24. In the first case, the floorbeams are supported by short posts; in the second they are connected directly

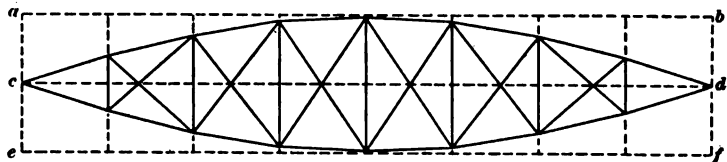


FIG. 24

to the vertical web members; and in the third they are suspended from the lower chord by hangers.

The graphic method by the stress diagram is the shortest for this truss as well as for the bowstring truss. In case it is desired to determine the stresses in the lenticular truss by the analytic method, the work can be considerably shortened by drawing the truss to a large scale, and scaling the lever arms of the web members about the various intersections of the chord members.

STRESSES IN BRIDGE TRUSSES

(PART 4)

MOMENTS AND SHEARS DUE TO CONCENTRATED LOADS

INTRODUCTION

1. The methods of finding the stresses due to uniformly distributed loads covering all or part of the span have been fully discussed. It is frequently necessary to determine the maximum stresses caused by a system of concentrated loads—such as the wheel loads of a locomotive and train of cars, street cars, road rollers, etc.—that moves over the span. The spacing of the wheels of such a system with respect to one another is fixed; the loads on the wheels are usually unequal, and the system is movable as a whole.

In designing trusses and plate girders, it is necessary to calculate the maximum moments and shears at several panel points, or sections along the span; in plate girders, it is also necessary to determine the section where the moment is greatest. This involves: (1) the determination of the position of the wheels that will cause the greatest moment or shear at any section; and (2) the actual calculation of the greatest moment or shear when the wheels occupy the required position. In practice, it is customary to consider the loads as moving from right to left, coming on the span at the right end and going off the span at the left end. This convention will be followed here.

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MAXIMUM MOMENTS AND SHEARS IN SIMPLE BEAMS

MAXIMUM MOMENT AT A GIVEN SECTION

2. A Single Concentrated Moving Load.—Let AB , Fig. 1 (a), be a simple beam over which a load W is moving, and let it be required to find the position of the load for which the moment at a given section C is a maximum. The distance AC will be denoted by a . When W is at any

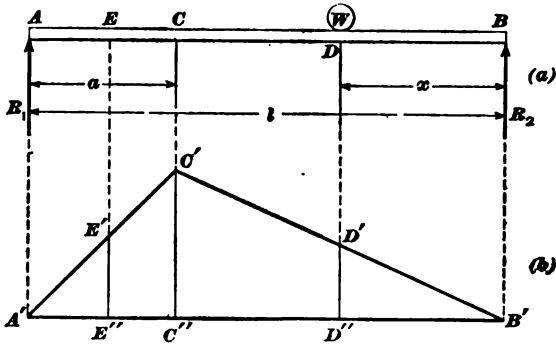


FIG. 1

point D to the right of C , and at a distance x from the right end, the reaction R_1 is $\frac{Wx}{l}$, and, therefore, the moment at C is $R_1 a$, or $\frac{Wx}{l} \times a$; as W , a , and l are constant, the moment at C is greatest when x is as great as possible. This occurs when x is equal to BC , or $l - a$; that is, when W is at C . When W is at the left of C at a distance x_1 from the left end, the moment at C is equal to $\frac{Wx_1}{l} \times (l - a)$, which is a maximum when $x_1 = a$, that is, when W is at C . It is thus seen that the maximum moment at C is equal to $\frac{W'a}{l} \times (l - a)$, and occurs when the load is at C . (See also *Strength of Materials*, Part 1.)

3. Influence Line for Moment.—The variation in the bending moment at C when W moves along the beam may be represented graphically as in Fig. 1 (*b*). The horizontal line $A'B'$ having been drawn at a convenient distance from AB , the vertical $C''C'$ is drawn, representing the bending moment at C when W is at C . Since the moment at C increases uniformly as the distance of W from the right end increases, the line $B'C'$, drawn straight from B' to C' , will represent the variation in moment as W moves from B to C ; also, the line $C'A'$, drawn straight from C' to A' , will represent the variation as W moves from C to A . For example, if $C''C'$ represents the moment at C when W is at C , then $D''D'$, vertically under D , represents the moment at C when W is at D ; also, $E''E'$ represents the moment at C when W is at E .

If the load W is made equal to unity (as 1 pound or 1 ton), and the ordinate $C''C'$ is made equal to the maximum moment at C for this load, the line $A'C'B'$ is called the **influence line** for the moment at C . Then, the moment at C due to any load W at D is equal to $W \times D''D'$; with a load W at E , the moment at C is $W \times E''E'$; etc. If there are loads W_1 and W_2 at the same time at E and D , respectively, the moment at C is equal to $W_1 \times E''E' + W_2 \times D''D'$, and similarly for any number of loads. The influence line is of great value in determining the position of a system of loads for which the moment or shear at a given section of a beam or truss, or the stress in a member, is a maximum.

4. Two Concentrated Loads.—Let AB , Fig. 2 (*a*), be a simple beam over which the loads W_1 and W_2 are moving. These loads are assumed to be at a fixed distance d apart. It is required to find the position of the loads for which the moment at a section C , distant a from the left end of the beam, is a maximum. The influence line for the moment at the section C is $A'C'B'$, Fig. 2 (*b*). Let y_1' and y_2' represent the ordinates under W_1 and W_2 , respectively. As the loads approach C from the right, both y_1' and y_2' increase until W_1 reaches C , when the moment is equal to

An influence line is a line showing variation in moment or shear at any given section of a beam or truss, or the stress in a member, as a unit load moves across it.

$W_1 y_1 + W_2 y_2$, Fig. 2, (c) and (d). As the loads move farther to the left, y_1 decreases and y_2 increases, and the moment at C increases or decreases according as $W_1 y_1$ decreases less or more than $W_2 y_2$ increases. When W_1 is at a distance b to the left of C , Fig. 2 (c), let y_1'' and y_2'' be the ordinates to the influence line corresponding to W_1 and W_2 ,

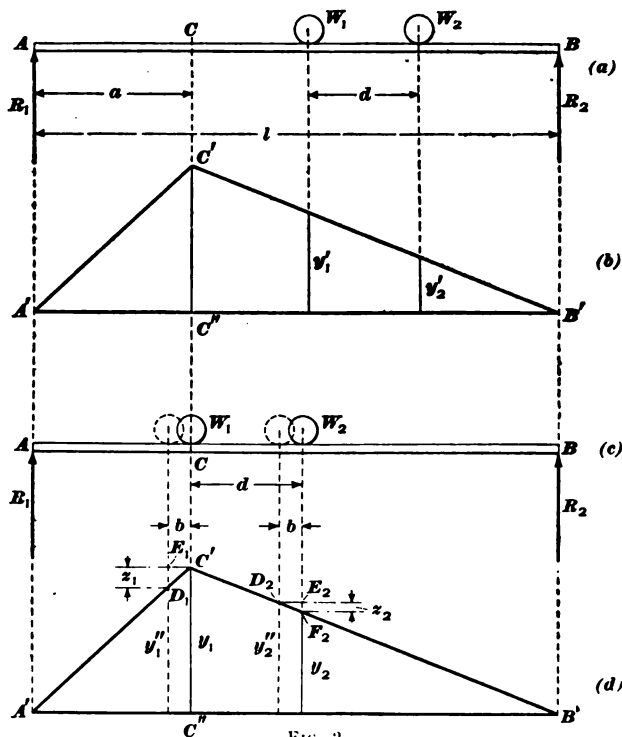


FIG. 2

respectively, and let z_1 be the difference between y_1 and y_1'' ; and z_2 , the difference between y_2 and y_2'' . Then, the total change in moment is equal to $+W_2 z_2 - W_1 z_1$. The similar triangles $A' C' C''$ and $D_1 E_1 C'$, Fig. 2 (d), give

$$D_1 E_1 : E_1 C' = C' C'' : A' C'';$$

that is,

$$1 : b = y_1 : A' C'';$$

whence

$$z_1 = b \times \frac{y_1}{A' C''}$$

Likewise, from the similar triangles $B' C' C''$ and $D_2 E_2 F_2$,

$$z_2 = b \times \frac{y_1}{B' C''}$$

Substituting these values for z_1 and z_2 , the change in moment becomes

$$W_2 \left(b \times \frac{y_1}{B' C''} \right) - W_1 \left(b \times \frac{y_1}{A' C''} \right) = b y_1 \left(\frac{W_2}{B' C''} - \frac{W_1}{A' C''} \right)$$

The moment at C will be increased if the change in moment is positive, and decreased if the change is negative; it is positive when $\frac{W_2}{B' C''}$ is greater than $\frac{W_1}{A' C''}$, and negative when

$\frac{W_2}{B' C''}$ is less than $\frac{W_1}{A' C''}$. In other words, the moment at C

will be increased if the right-hand load divided by the right-hand portion of the span is greater than the left-hand load divided by the left-hand portion of the span, and decreased if the former quotient is less than the latter. Then, if the change in moment is negative, the maximum moment obtains when W_1 is at C ; if positive, the moment will increase until W_2 reaches C , when it is a maximum; for, when W_2 passes C , both y_1 and y_2 , and, therefore, the moment at C , decrease. Hence the following principle:

The maximum bending moment caused at any given section of a beam by two concentrated moving loads obtains when one of the loads is at the section. If the right-hand load divided by the length of the right-hand portion of the beam is greater than the left-hand load divided by the length of the left-hand portion of the beam, the moment is a maximum when the right-hand load is at the section; if the left-hand load divided by the left-hand portion of the beam is the greater, the moment is a maximum when the left-hand load is at the section.

If the loads are equal, this principle will always bring the left-hand load at the section when the section is at the left of the center, and the right-hand load when the section is at the right of the center.

To find the value of the bending moment at C , the left reaction R , may first be found by taking moments about B .

Then, if W_1 is at C , the moment is equal to $R_1 \times a$; if W_2 is at C , the moment is equal to $R_1 \times a - W_1 \times d$.

EXAMPLE.—A beam 40 feet long supports a system of two concentrated loads 8 feet apart. The left-hand load is 10,000 pounds, and the right-hand load is 15,000 pounds. What are the maximum moments: (a) at a section 10 feet from the left end? (b) at the center?

SOLUTION.—(a) The length of the left-hand portion of the beam is 10 ft.; that of the right-hand portion is 30 ft. Then,

$$10,000 \div 10 = 1,000 \text{ and } 15,000 \div 30 = 500$$

As the first is the greater, the moment is a maximum when the left-hand load is at the section. Then,

$$R_1 = \frac{10,000 \times 30 + 15,000 \times 22}{40} = 15,750 \text{ lb.}$$

and the bending moment is equal to

$$15,750 \times 10 = 157,500 \text{ ft.-lb. Ans.}$$

(b) The left-hand and right-hand portions have each a length of 20 ft. Then,

$$10,000 \div 20 = 500 \text{ and } 15,000 \div 20 = 750$$

As the second is the greater, the moment is a maximum when the right-hand load is at the center. Then,

$$R_1 = \frac{10,000 \times 28 + 15,000 \times 20}{40} = 14,500 \text{ lb.}$$

and the bending moment is equal to

$$14,500 \times 20 - 10,000 \times 8 = 210,000 \text{ ft.-lb. Ans.}$$

70 here

5. Any Number of Concentrated Loads.—When the system consists of more than two loads, the method of treatment is practically the same as for two loads; the moment at a given section is a maximum when one of the loads is at the section. Let AB , Fig. 3 (a), be a simple beam over which a system of concentrated loads is moving, and let it be required to find the position of the loads when they cause the greatest moment at C , at a distance a from the left end. Let $A'C'B'$, Fig. 3 (b), be the influence line for the moment at C , and let y_1, y_2 , etc. be the ordinates corresponding to W_1, W_2, W_3 , etc., respectively. As the loads move across the beam and W_1 approaches C , the bending moment at C increases until W_1 reaches C , when it is equal to $W_1 y_1 + W_2 y_2 + \dots + W_n y_n$. When W_1 passes C , the moment due to it decreases, and that due to the remaining loads

increases; it can be shown, as in Art. 4, that the change in moment is equal to

$$b \times y_1 \left(\frac{W_2 + W_3 + \dots + W_n}{B' C''} - \frac{W_1}{A' C''} \right)$$

If $\frac{W_2 + W_3 + \dots + W_n}{B' C''}$ is less than $\frac{W_1}{A' C''}$, the moment decreases after W_1 passes C , and is therefore greatest

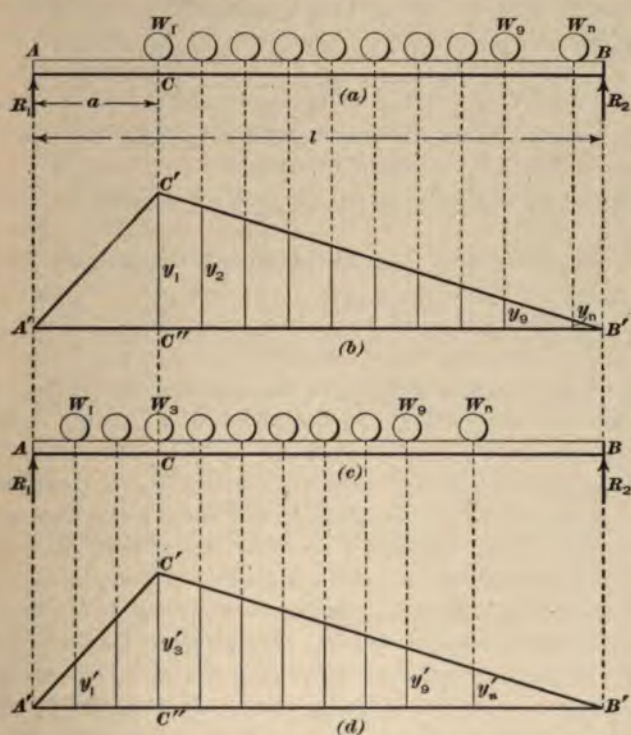


FIG. 8

when W_1 is at C ; if the former of these two fractions is greater than the latter, the moment increases until W_2 reaches C . When W_2 passes C , the change in moment is equal to

$$b \times y_1 \left(\frac{W_3 + W_4 + \dots + W_n}{B' C''} - \frac{W_1 + W_2}{A' C''} \right)$$

If $\frac{W_1 + W_2 + \dots + W_n}{B'C''}$ is less than $\frac{W_1 + W_2}{A'C''}$, the moment decreases after W_1 passes C , and is therefore greatest when W_1 is at C ; if the former of these two fractions is greater than the latter, the moment increases until W_2 reaches C , etc. Suppose, in this case, that the moment increases after W_1 and W_2 pass C , and decreases after W_3 passes C ; then, the moment is greatest when W_3 is at C [see Fig. 3. (c) and (d)]. Then,

$$\frac{W_1 + W_2}{A'C''} \text{ is less than } \frac{W_3 + W_4 + \dots + W_n}{B'C''}$$

from the condition that the moment increases after W_1 passes C until W_3 reaches C ; and

$$\frac{W_1 + W_2 + W_3}{A'C''} \text{ is greater than } \frac{W_4 + W_5 + \dots + W_n}{B'C''}$$

from the condition that the moment decreases after W_3 passes C .

The quotient obtained by dividing the sum of all the loads on a part of a beam by the length of that part will be referred to as the **average intensity of load** on that part of the beam.

From the preceding discussion, the following general principle may be stated:

The maximum bending moment caused at a given section of a beam by a system of moving concentrated loads obtains when such a load is at the section that, when it is counted with the loads on the right, the average intensity of load on the right is greater than the average intensity of load on the left; and when it is counted with the loads on the left, the average intensity of load on the left is greater than the average intensity of load on the right.

When the loads are in this position, if part of the load at the section is counted with the loads on the left and the sum is called W_l , and the remainder counted with the loads on the right and the sum called W_r , it is possible to make $\frac{W_l}{a}$

equal to $\frac{W_r}{l-a}$. Putting $\frac{W_l}{a} = \frac{W_r}{l-a}$, and denoting the sum of all the loads on the span by ΣW , we have

$$W_r = \Sigma W - W_l,$$

and, therefore, $\frac{W_i}{a} = \frac{\Sigma W - W_i}{l - a},$

whence $W_i \times \frac{l - a}{a} = \Sigma W - W_i,$

or $W_i \left(\frac{l}{a} - 1 \right) = \Sigma W - W_i,$

and $W_i \times \frac{l}{a} = \Sigma W, \frac{W_i}{a} = \frac{\Sigma W}{l}, W_i = \frac{a}{l} \times \Sigma W$

The principle just given may therefore be stated more simply as follows:

The maximum bending moment caused at a given section of a beam by a system of moving concentrated loads obtains when such a load is at the section that, if part of it is counted with the loads on the left, the average intensity of load on the left is equal to that on the whole beam.

As W_i is equal to the sum of all the loads to the left of C plus a part of the load at C , the sum of all the loads to the left of C must be less than W_i , or $\frac{a}{l} \times \Sigma W$, and this sum

added to the load at C must be greater than W_i , or $\frac{a}{l} \times \Sigma W$.

If the system is such that all the loads are on the beam when the moment is a maximum, W_i may be found directly from the formula $W_i = \frac{a}{l} \times \Sigma W$. If the system is such that some

of the loads are off the beam when the moment is a maximum, the correct position may be found by trial by placing several of the heaviest loads at the section successively to see which fulfils the condition. It will frequently be found that several positions of the loads will satisfy the conditions for a maximum (on account of the fact that loads pass off at the left end and on at the right end as the loads move to the left to occupy the different positions); in this case, the actual value of the moment must be calculated for each position that satisfies the conditions, in order to determine which position causes the greatest moment. A load that comes exactly at A or B when one load is placed at C should be considered as off the span.

6. Value of the Moment.—If Wx represents the moment about B of any load at a distance x from the right end, the moment of all the loads on the span about B may be denoted by $\sum Wx$, and the left reaction R_1 by $\frac{\sum Wx}{l}$. If Wb represents the moment about C of any load to the left of C at a distance b from C , the moment of all the loads on the left end about C may be written $\sum Wb$. Then, the bending moment at C is $R_1 a - \sum Wb$; that is, $\frac{a}{l} \times \sum Wx - \sum Wb$.

EXAMPLE 1.—A beam 50 feet long supports the system of moving concentrated loads represented in Fig. 4. To find the maximum moment: (a) at a point 10 feet from the left end; (b) at the center.

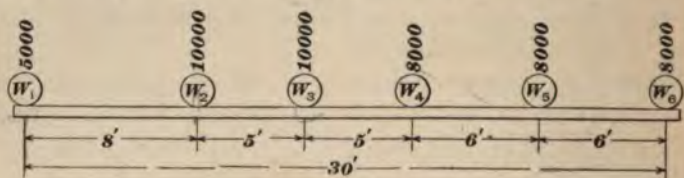


FIG. 4

SOLUTION.—(a) As the system of loads is much shorter than the beam, it may be assumed that all the loads will be on. Then, $\sum W = 49,000$ lb.; $W_l = \frac{a}{l} \times \sum W = \frac{10}{50} \times 49,000 = 9,800$ lb.; $W_1 = 5,000$ lb.; and $W_1 + W_2 = 15,000$ lb. As one value is less and the other greater than 9,800 (Art. 5), the moment is a maximum when W_2 is at C . Then the distances of the various loads from the right end are 48, 40, 35, 30, 24, and 18 ft., respectively; and $R_1 = (5,000 \times 48 + 10,000 \times 40 + 10,000 \times 35 + 8,000 \times 30 + 8,000 \times 24 + 8,000 \times 18) \div 50 = 31,320$ lb.

The bending moment is equal to

$$31,320 \times 10 - 5,000 \times 8 = 273,200 \text{ ft.-lb. Ans.}$$

(b) In this case, $W_l = \frac{25}{50} \times 49,000 = 24,500$ lb. The sum of W_1

and W_2 is 15,000 lb.; of W_1 , W_2 , and W_3 , 25,000 lb. As one value is less and the other greater than 24,500, the moment is a maximum when W_3 is at C . Then, the distances of the loads from the right end are 38, 30, 25, 20, 14, and 8 ft., respectively; and

$$R_1 = (5,000 \times 38 + 10,000 \times 30 + 10,000 \times 25 + 8,000 \times 20 + 8,000 \times 14 + 8,000 \times 8) \div 50 = 21,520 \text{ lb.}$$

The bending moment is equal to

$$21,520 \times 25 - (5,000 \times 13 + 10,000 \times 5) = 423,000 \text{ ft.-lb. Ans.}$$

EXAMPLE 2.—With the same system of loads as in the preceding example, what is the greatest moment at the center of a beam 25 feet long?

SOLUTION.—In this case, the system of loads is longer than the beam, and some loads will be off; the position that causes the greatest moment will be determined largely by trial.

When W_1 is at the center, W_1 and W_2 are on the beam. Then,

$$W_l = \frac{a}{l} \times \Sigma W = \frac{12.5}{25} \times 15,000 = 7,500 \text{ lb.}$$

As this is larger than W_1 , the moment is not a maximum when W_1 is at the center.

When W_2 is at the center, W_1 , W_2 , W_3 , and W_4 are on the beam. Then,

$$W_l = \frac{12.5}{25} \times 33,000 = 16,500 \text{ lb.}$$

As this is larger than $W_1 + W_2$ (15,000 lb.), the moment is not a maximum when W_2 is at the center.

When W_3 is at the center, W_2 , W_3 , W_4 , and W_5 are on the beam. Then,

$$W_l = \frac{12.5}{25} \times 36,000 = 18,000 \text{ lb.}$$

As this is larger than $W_2 + W_3$ (20,000 lb.), and smaller than $W_3 + W_4$ (20,000 lb.), the moment is a maximum when W_3 is at the center.

When W_4 is at the center, W_3 , W_4 , W_5 , and W_6 are on the beam. Then,

$$W_l = \frac{12.5}{25} \times 44,000 = 22,000 \text{ lb.}$$

As this is larger than $W_3 + W_4$ (20,000 lb.), and smaller than $W_4 + W_5 + W_6$ (28,000 lb.), the moment is also a maximum when W_4 is at the center.

When W_5 is at the center, W_4 , W_5 , and W_6 are on the beam. Then,

$$W_l = \frac{12.5}{25} \times 34,000 = 17,000 \text{ lb.}$$

As this is less than $W_4 + W_5$ (18,000 lb.), the moment is not a maximum when W_5 is at the center.

There are then two positions that satisfy the conditions for maximum moment; namely, when W_3 is at the center, and when W_4 is at the center. It is necessary to calculate both values.

When W_3 is at the center,

$$R_1 = \frac{10,000 \times 17.5 + 10,000 \times 12.5 + 8,000 \times 7.5 + 8,000 \times 1.5}{25} \\ = 14,880 \text{ lb.}$$

The bending moment is

$$14,880 \times 12.5 - 10,000 \times 5 = 136,000 \text{ ft.-lb.}$$

When W_4 is at the center,

$$R_1 = (10,000 \times 22.5 + 10,000 \times 17.5 + 8,000 \times 12.5 + 8,000 \times 6.5 + 8,000 \times 0.5) \div 25 = 22,240 \text{ lb.}$$

The bending moment is

$$22,240 \times 12.5 - (10,000 \times 10 + 10,000 \times 5) = 128,000 \text{ ft.-lb.}$$

Therefore, the moment at the center is greatest when W_4 is at the center, and is equal to 136,000 ft.-lb. Ans.

EXAMPLES FOR PRACTICE

1. A beam 50 feet long supports a moving load consisting of two concentrations 10 feet apart. The left-hand load is 12,000 pounds, and the right-hand load is 18,000 pounds. What is the maximum moment: (a) at a section 10 feet from the left end? (b) at the center? (c) at a section 40 feet from the left end?

$$\text{Ans. } \begin{cases} (a) & 204,000 \text{ ft.-lb.} \\ (b) & 315,000 \text{ ft.-lb.} \\ (c) & 216,000 \text{ ft.-lb.} \end{cases}$$

2. A beam 50 feet long supports a moving load consisting of two concentrations of 10,000 pounds each at a distance of 8 feet apart. What is the maximum moment at a section 20 feet from the left end?

$$\text{Ans. } 208,000 \text{ ft.-lb.}$$

3. A beam 35 feet long supports a moving load consisting of the system shown in Fig. 4. What is the greatest moment: (a) at a section 10 feet from the left end? (b) at the center?

$$\text{Ans. } \begin{cases} (a) & 203,100 \text{ ft.-lb.} \\ (b) & 239,250 \text{ ft.-lb.} \end{cases}$$

4. A beam 30 feet long supports a moving load consisting of the

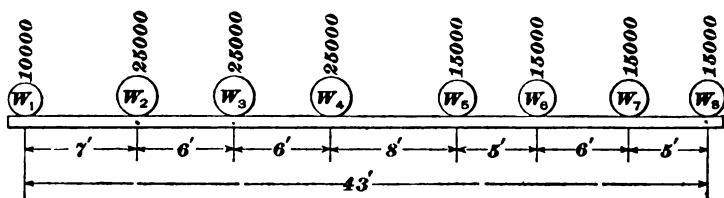


FIG. 5

system shown in Fig. 5. What is the greatest moment: (a) at a section 25 feet from the left end? (b) at the center?

$$\text{Ans. } \begin{cases} (a) & 247,500 \text{ ft.-lb.} \\ (b) & 430,000 \text{ ft.-lb.} \end{cases}$$

SECTION OF MAXIMUM MOMENT IN A BEAM

7. Any Number of Loads.—As a system of concentrated loads moves across a beam, there is a section where, under a certain position of the loads, the moment is greater than occurs at any other section under any position of the loads. The method of determining the location of this section will now be considered.

Let AB , Fig. 6, be a simple beam over which a system of concentrated loads is moving, and let it be required to find the section where the moment is a maximum as the loads move across the beam. The greatest bending moment will occur under one of the loads, and it is necessary to determine the position of the system for which the bending moment under any one of the given loads is a maximum.

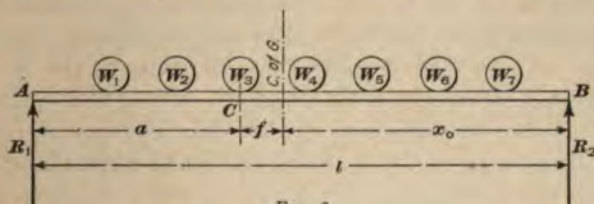


FIG. 6

Let C be the position of W_3 when the bending moment under W_3 is a maximum, and let $AC = a$. The left reaction R_1 is $\frac{\sum Wx}{l}$, denoting by x the distance of any load from the right-hand support. If $\sum W$ represents the sum of all the loads on the beam, and x_0 the distance from the right end to the center of gravity of these loads, then

$$\sum Wx = x_0 \sum W,$$

and, therefore, $R_1 = \frac{x_0 \sum W}{l}$. The sum of the moments of the loads on the left end about C is $\sum Wb$, denoting by b the distance of any of those loads for C . The bending moment at C is equal to $R_1 \times a - \sum Wb$, or $\frac{ax_0 \sum W}{l} - \sum Wb$.

This expression is a maximum when ax_0 is a maximum; that is, when $x_0 = a$, or when C is the same distance

from A as the center of gravity of the loads is from B ; in other words, when C and the center of gravity of the loads are equidistant from the center of the beam.* Whence the principle:

The maximum moment under a load occurs when that load and the center of gravity of all the loads on the beam are equidistant from the center.

This principle, in connection with the principle that the loads on the left of the section of maximum moment must satisfy the condition $W_l = \frac{a}{l} \times \Sigma W$ (Art. 5), serves to determine the greatest bending moment in the beam.

In applying the preceding principles, the required position of the loads must be determined largely by trial. If the system of loads is longer than the beam, some of the loads will be off when the moment is a maximum. In such a case, it will be well to remember that the section of maximum moment will usually be close to the center; the position of the loads, when the moment at the center is a maximum, may be found as explained in Art. 5. The position of the center of gravity of the loads on the beam for this condition may be calculated, and the loads placed according to the principle just given, placing them first so that the center of the span is half way between the center of gravity and the nearest load, then the next nearest, etc., until a position is found that satisfies all the conditions for maximum moment. If, when the loads occupy the required position, there are more or less on the beam than those for which the center of gravity was computed, it will be necessary to compute anew the location of the center of gravity of the loads then on the beam, and rearrange them. In case more than one position of the loads satisfies the conditions for the maximum bending

*Let the distance from the center of gravity of the loads to C be represented by f , Fig. 6; then, $x_0 + a = l - f$, which may be called c ; also, $x_0 = c - a$, and $a x_0 = a(c - a) = ac - a^2 = \left(\frac{c}{2}\right)^2 - \left(a - \frac{c}{2}\right)^2$, and this is a maximum when the negative term is zero; that is, when $a - \frac{c}{2} = 0$; whence $a = \frac{c}{2} = \frac{l - f}{2} = \frac{x_0 + a}{2}$, and, therefore, $x_0 = a$.

moment, it will be necessary to compute the bending moment for each position in order to find the greatest moment.

8. Value of the Moment.—In Art. 7 it was stated that the bending moment at C is equal to $\frac{ax_0 \sum W}{l} - \sum Wb$.

For the maximum bending moment M under a load, we have, since in this case $x_0 = a$,

$$M = \frac{x_0^2 \sum W}{l} - \sum Wb$$

EXAMPLE 1.—A beam 50 feet long supports the system of moving concentrated loads shown in Fig. 4. What is the location of the section of maximum bending moment, and the value of the moment?

SOLUTION.—As this system of loads is much shorter than the beam, it may be assumed that all the loads will be on when the moment is greatest. The center of gravity may be found by taking moments about W_1 ; its distance from W_1 is

$$\frac{5,000 \times 30 + 10,000 \times 22 + 10,000 \times 17 + 8,000 \times 12 + 8,000 \times 6 + 8,000 \times 0}{49,000} = 13.96 \text{ ft.}$$

The center of gravity is 1.96 ft. to the left of W_4 and 3.04 ft. to the right of W_2 . As W_4 is nearer, it will be tried first.

When W_4 and the center of gravity are equidistant from the center,

$$x_0 = a = 25 + \frac{1.96}{2} = 25.98 \text{ ft.}$$

and $\frac{a}{l} \times \sum W = \frac{25.98}{50} \times 49,000 = 25,500 \text{ lb.}$

$W_1 + W_2 + W_3 = 25,000 \text{ lb.}$; $W_1 + W_2 + W_3 + W_4 = 33,000 \text{ lb.}$ As one of these sums is less and the other greater than 25,500 lb., the moment is a maximum under W_4 .

The bending moment is

$$\frac{25.98^2 \times 49,000}{50} - (5,000 \times 18 + 10,000 \times 10 + 10,000 \times 5) = 421,500 \text{ ft.-lb.}$$

When W_2 and the center of gravity are equidistant from the center,

$$x_0 = a = 25 - \frac{3.04}{2} = 23.48 \text{ ft.}$$

and $\frac{a}{l} \times \sum W = \frac{23.48}{50} \times 49,000 = 23,000 \text{ lb.}$

$W_1 + W_2 = 15,000 \text{ lb.}$; $W_1 + W_2 + W_3 = 25,000 \text{ lb.}$ As one of these sums is less and the other greater than 23,000 lb., the moment is a maximum under W_3 .

The bending moment is

$$\frac{23.48^2 \times 49,000}{50} - (5,000 \times 13 + 10,000 \times 5) = 425,300 \text{ ft.-lb.}$$

If any other loads are tried, it will be found that they do not satisfy the condition for maximum moment. The section of maximum moment is, therefore, 23.48 ft. from the left end, and the bending moment at that section is 425,000 ft.-lb. Ans.

EXAMPLE 2.—A beam 25 feet long supports the system of moving loads shown in Fig. 4. What is the location of the section of maximum moment, and the value of the moment?

SOLUTION.—As the system of loads is longer than the beam, some of the loads will be off the span when the moment is a maximum. In example 2, Art. 6, it was shown that the condition for maximum moment at the center of this span for the same load is satisfied when W_1 is at the center, and when W_5 is at the center; it will, therefore, be unnecessary to repeat the demonstration here.

When W_1 is at the center, W_1 , W_2 , W_3 , W_4 , and W_5 are on the beam, and the distance of the center of gravity of these loads from W_1 is

$$\frac{10,000 \times 16 + 10,000 \times 11 + 8,000 \times 6 + 8,000 \times 0}{36,000} = 8.83 \text{ ft.}$$

The center of gravity is 2.83 ft. to the left of W_1 , and 2.17 ft. to the right of W_5 . As W_5 is nearer, it may be tried first.

$$x_5 = a = 12.5 - \frac{2.17}{2} = 11.42 \text{ ft.}$$

$$\text{Then, } \frac{a}{l} \times \Sigma W = \frac{11.42}{25} \times 36,000 = 16,400 \text{ lb.}$$

As this is greater than W_1 and less than $W_1 + W_5$, the moment is a maximum under W_5 , and is equal to

$$\frac{11.42^2 \times 36,000}{25} - 10,000 \times 5 = 137,800 \text{ ft.-lb.}$$

When W_1 and the center of gravity are equidistant from the center,

$$x_5 = a = 12.5 - \frac{2.83}{2} = 13.92 \text{ ft.}$$

$$\text{and } \frac{a}{l} \times \Sigma W = \frac{13.92}{25} \times 36,000 = 20,000 \text{ lb.}$$

As this is less than $W_1 + W_5 + W_4$, and equal to $W_1 + W_5$, the moment is a maximum under W_4 , and is equal to

$$\frac{13.92^2 \times 36,000}{25} - 10,000 \times 10 - 10,000 \times 5 = 129,000 \text{ ft.-lb.}$$

The fact that $\frac{a}{l} \times \Sigma W$ is equal to $W_1 + W_5$ in this case means that, after W_1 passes C , the moment neither increases nor decreases until W_5 reaches C , after which it decreases.

When W_1 is at the center, W_1 , W_2 , W_3 , W_4 , and W_5 are on the beam, and the center of gravity of these loads is

$$\frac{10,000 \times 22 + 10,000 \times 17 + 8,000 \times 12 + 8,000 \times 6 + 8,000 \times 0}{44,000} = 12.14 \text{ ft.}$$

from W_1 , or .14 ft. to the left of W_4 ; then C will be .07 ft. to the right

of the center, and for all practical purposes it may be assumed to be at the center. The moment at the center under W_1 has already been found (example 2, Art. 6) to be equal to 128,000 ft.-lb. It is unnecessary to try any other positions.

From the foregoing, it is seen that the maximum moment occurs under W_2 , when that load is 11.42 ft. from the left end, and the moment is equal to 137,800 ft.-lb. Ans.

9. Comparison.—In example 2, Art. 6, it was found that the maximum moment at the center of a 25-foot beam for the same loading as in the preceding example is about 1.3 per cent. less than the actual maximum moment just found; and in example 1, Art. 6, it was found that the maximum moment at the center of a 50-foot beam for the same loading is about .5 per cent. less than the actual maximum moment just found. In any case, the actual maximum moment is very little larger than that at the center, and, for all spans above 50 feet in length, with the ordinary conditions of loading, the latter is sufficiently accurate for all practical purposes. For spans up to about 50 feet, it may be desirable to compute the actual maximum.

10. Special Cases.—The following principles are but special cases of the general principle stated in Art. 8:

1. *The maximum moment in a beam supporting two unequal loads will occur under the heavier load when that load and the center of gravity of the two loads are equidistant from the center of gravity of the beam.*

2. *The maximum moment in a beam supporting two equal loads will occur under either load when the two loads are on opposite sides of the center and one of the loads is at a distance from the center equal to one-fourth the distance between the loads.*

3. *In case a beam that supports two loads is but little longer than the distance between the loads, it may happen that one of the loads placed at the center of the beam will cause a greater moment than if the two loads are arranged as described in the first principle.*

EXAMPLE 1.—A beam 24 feet long supports two moving loads 6 feet apart; the left-hand load is 8,000 pounds, and the right-hand load is 4,000 pounds. What is the location of the section of maximum moment, and what is the value of the moment?

SOLUTION.—The center of gravity is

$$\frac{8,000 \times 6}{12,000} = 4 \text{ ft.}$$

from the right-hand load. Then the section of maximum moment is 1 ft. to the left of the center; therefore, $a = x_0 = 11$ ft., and the moment is equal to

$$\frac{11^2 \times 12,000}{24} = 60,500 \text{ ft.-lb. Ans.}$$

EXAMPLE 2.—A beam 25 feet long supports two equal moving loads of 10,000 pounds each, 6 feet apart. What is the location of the section of maximum moment, and what is the value of the moment?

SOLUTION.—As the beam supports two equal moving loads, the second principle applies in this case. One-fourth the distance between the loads is $6 \div 4 = 1.5$ ft., and the section of maximum moment is therefore under either load when that load is 1.5 ft. from the center of the beam. At this section,

$$a = x_0 = \frac{25}{2} - 1.5 = 11 \text{ ft.,}$$

and the moment is equal to

$$\frac{11^2 \times 20,000}{25} = 96,800 \text{ ft.-lb. Ans.}$$

EXAMPLE 3.—A beam 8 feet long supports two moving loads 5 feet apart. The left-hand load is 6,000 pounds, and the right-hand load is 4,000 pounds. What is the location of the section of maximum moment, and what is the value of that moment?

SOLUTION.—The distance of the center of gravity of the two loads from the right-hand load is equal to

$$\frac{6,000 \times 5}{6,000 + 4,000} = 3 \text{ ft.}$$

and the distance from the left-hand load is $5 - 3 = 2$ ft. Applying the first principle for a maximum moment, the load of 6,000 lb. will be $2 \div 2 = 1$ ft. to the left of the center of the beam, and $\frac{8}{2} - 1 = 3$ ft. from the left end of the beam. That is, $a = x_0 = 3$ ft., and the moment is

$$\frac{3^2 \times 10,000}{8} = 11,250 \text{ ft.-lb.}$$

As the beam is but little longer than the distance between the loads, according to the third principle it may happen that the maximum moment will occur when one of the loads is at the center of the beam. When the load of 6,000 lb. is at the center, the load of 4,000 lb. will be off the beam, and the maximum moment will be equal to

$$\frac{4^2 \times 6,000}{8} = 12,000 \text{ ft.-lb.}$$

As this value is greater than the one just found, the section of maximum moment is at the center, and the moment is 12,000 ft.-lb. Ans.

EXAMPLES FOR PRACTICE

1. A beam 60 feet long supports the system of moving concentrated loads shown in Fig. 5. (a) What is the location of the section of maximum moment? (b) What is the value of the maximum moment?

Ans. $\begin{cases} (a) & \text{At 28.90 ft. from the left end} \\ (b) & 1,378,400 \text{ ft.-lb.} \end{cases}$

2. A beam 30 feet long supports the system of moving concentrated loads shown in Fig. 5. (a) What is the location of the section of maximum moment? (b) What is the value of that moment?

Ans. $\begin{cases} (a) & \text{At 14.6 ft. from the left end} \\ (b) & 430,500 \text{ ft.-lb.} \end{cases}$

3. A beam 30 feet long supports two moving loads 8 feet apart. If the left-hand load is 30,000 pounds, and the right-hand load is 25,000 pounds, find: (a) the location of the section of maximum moment; (b) the value of the moment.

Ans. $\begin{cases} (a) & \text{At 13.18 ft. from the left end} \\ (b) & 318,500 \text{ ft.-lb.} \end{cases}$

4. Two loads 6 feet apart are moving over a beam 10 feet long. If the left-hand load is 20,000 pounds, and the right-hand load is 10,000 pounds: (a) what is the location of the section of maximum moment? (b) what is the value of that moment?

Ans. $\begin{cases} (a) & \text{At the center} \\ (b) & 50,000 \text{ ft.-lb.} \end{cases}$

MAXIMUM SHEAR AT A GIVEN SECTION

11. Single Moving Load.—Let AB , Fig. 7 (a), be a

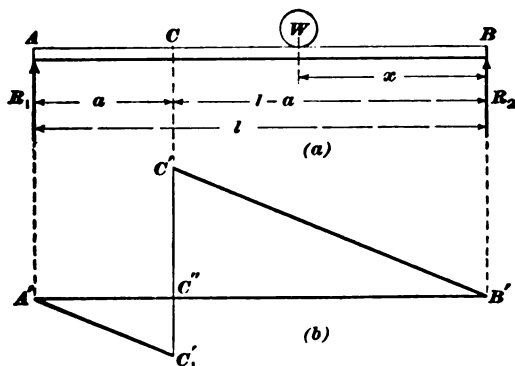


FIG. 7

simple beam supporting the moving load W , and let it be required to find the position of the load for which the shear

at the section C , at a distance a from the left end, is a maximum. When W is at any point to the right of C , at a distance x from the right end, the shear at C is positive and equal to the left-hand reaction $\frac{Wx}{l}$; when W is at any point to the left of C , the shear at C is negative, and equal to $\frac{Wx}{l} - W = -\frac{W(l-x)}{l}$. That is, as W passes on the beam at B and moves toward C , the shear at C increases uniformly until W is at C ; as W passes C , the shear suddenly changes from positive to negative; and, as W approaches A , the shear uniformly approaches zero. The shear at C is, then, a maximum when W is at C .

12. Influence Line for Shear.—The variation in shear at C as W moves over the beam may be represented graphically, as in Fig. 7 (*b*). The horizontal line $A'B'$ having been drawn at any convenient distance from AB , the vertical line $C''C'$ is laid off above $A'B'$, equal, by scale, to $\frac{W(l-a)}{l}$, and $C''C'_1$ is laid off below $A'B'$ equal to $\frac{Wa}{l}$; then, $B'C'$ represents the variation in shear at C as W moves from B to C , and C'_1A' represents the variation as W moves from C to A . If the ordinates $C''C'$ and $C''C'_1$ are made equal to the shear at C due to a load of unity, the line $B'C'C'_1A'$ is called the **Influence line** for the shear at C .

13. Any Number of Loads.—Let AB , Fig. 8 (*a*), be a beam supporting a system of moving concentrated loads, and let it be required to find the position of the loads when they cause maximum shear at the section C at a distance a from the left end. As the loads come on the beam at B and move toward the left, the shear at C increases until W_1 reaches C ; it is then equal to R_1 , and may be denoted by I_1 . If the sum of the loads then on the beam is ΣW , and the distance from their center of gravity to the right end is x , then $R_1 = I_1 = \frac{x}{l} \times \Sigma W$. When W_1 passes C , the shear suddenly decreases, and, when W_1 is at a distance k to the

left of C , is equal to $\frac{x_0 + k}{l} \times \sum W - W_1$. As the loads move farther to the left, k , and, therefore, the shear at C , increase until W_1 is at C . If d_1 is the distance from W_1 to

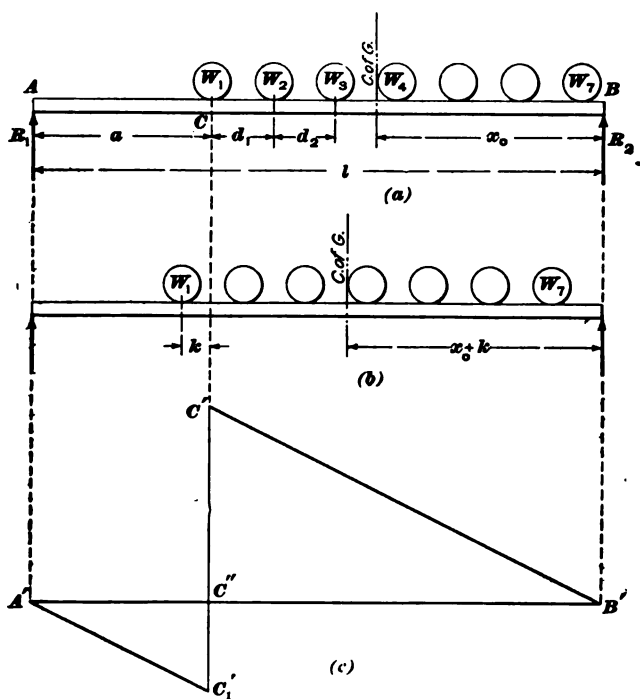


FIG. 8

W_1 , the shear when W_1 is at C , which may be called V_1 , is equal to $\frac{x_0 + d_1}{l} \times \sum W - W_1$, the difference between V_1 and V_1' being $\frac{d_1}{l} \times \sum W - W_1$. If this difference is negative, V_1' is less than V_1 , and the shear is a maximum when W_1 is at C ; if positive, V_1' is greater than V_1 . Suppose, in this case, that the shear is greater when W_1 is at C than when W_1 is at C . Then, when W_1 passes C , the shear again suddenly decreases, and, when W_1 is at a distance k to the left of C , is

equal to $\frac{x_0 + d_1 + k}{l} \times \Sigma W - (W_1 + W_n)$; as the loads move to the left, k increases, and, therefore, the shear increases until W_1 is at C . If d_1 is the distance from W_1 to W_n , the shear at C when W_1 is at C , which may be called V_1 , is equal to $\frac{x_0 + d_1 + d_1}{l} \times \Sigma W - (W_1 + W_n)$, the difference between V_1 and V_2 being $\frac{d_1}{l} \times \Sigma W - W_n$. If this difference is negative, V_2 is less than V_1 , and the shear is a maximum when W_1 is at C ; if positive, V_2 is greater than V_1 . By proceeding in this way, it will be found that, when some load W_n is at C , the shear V_n is less than the shear V_{n-1} , when the preceding load W_{n-1} is at C . Then, the shear at C is a maximum when W_{n-1} is at C . By trying the first two or three loads of a system successively at the section, the position of the loads for maximum shear is readily found. As all loads to the left of the section decrease the shear, there will usually be very few, if any, in that position.

14. Special Conditions.—In case some of the loads pass off the beam or others come on as the loads move to

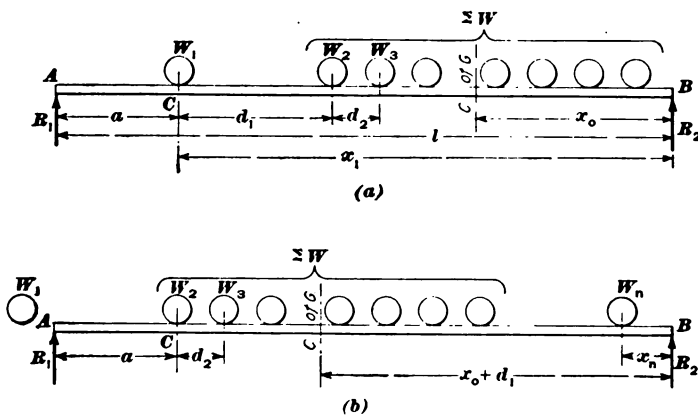


FIG. 9

the left, the value of ΣW will change. The manner of treating such cases will be shown by the following two illustrations.

Starting with W_1 at C , Fig. 9 (a), let it be assumed that, when the loads move a distance d_1 to the left, W_1 passes off the beam and W_n comes on [Fig. 9 (b)]. Taking ΣW as the sum of all the loads, exclusive of W_1 and W_n , the increase in shear is $\frac{d_1}{l} \times \Sigma W + \frac{W_n x_n}{l}$. The load W_1 caused a positive shear at C equal to $\frac{W_1 x_1}{l}$ when it was at C ; its effect after the loads move a distance d_1 to the left is zero; then the decrease in shear is $\frac{W_1 x_1}{l}$. The total increase is, therefore, $\frac{d_1}{l} \times \Sigma W + \frac{W_n x_n}{l} - \frac{W_1 x_1}{l}$; if this expression is negative, the shear decreases, and is a maximum when W_1 is at C ; if positive, the shear increases, and is greater when W_n is at C than when W_1 is at C .

Starting with W_2 at C , Fig. 10 (a), let it be assumed that W_1 is still on the beam but passes off when the loads

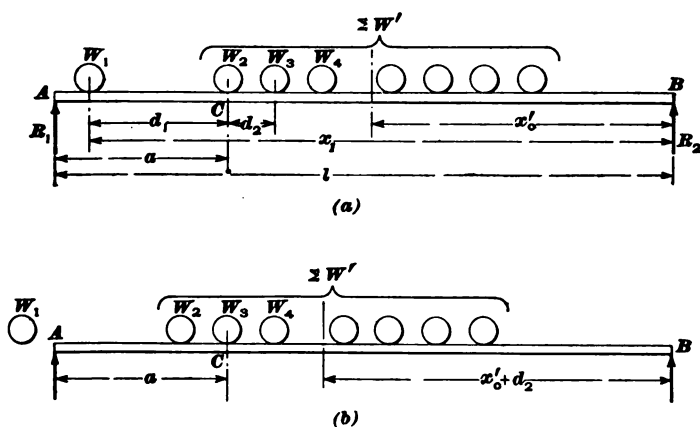


FIG. 10

move to the left, and that, when W_2 is at C , W_1 is still on the beam, no loads coming on at B [Fig. 10 (b)]. Here, $\Sigma W'$ will be taken as the sum of all the loads exclusive of W_1 . The shear when W_2 is at C is $\frac{x'_0}{l} \times \Sigma W' + \frac{W_2 x_2}{l} - W_1$, and,

when W_2 is at C , it is $\frac{x_1' + d_2}{l} \times \Sigma W' - W_2$, the difference being $\frac{d_2}{l} \times \Sigma W' - W_2 + \frac{W_1(l - x_1)}{l}$. If this difference is negative, the shear is decreased; if positive, it is increased.

The student should familiarize himself with the principles just outlined, and not try to remember the expressions as formulas, as each case that arises will probably be somewhat different from those here assumed.

15. Value of the Shear.—When the position of the loads has been determined, it is easy to calculate the value of the shear. If Wx represents the moment of any one of the loads about the right end, and W_l the sum of all the loads on the left of the section, the shear is equal to

$$\frac{\Sigma Wx}{l} - W_l$$

16. Section of Maximum Shear.—The shear is a maximum at the section where the value of $\frac{\Sigma Wx}{l} - W_l$ is greatest; this is just to the right of the left end, and for all practical purposes may be taken at the left end; at that section, W_l is zero and ΣWx is a maximum.

EXAMPLE.—A beam 40 feet long supports the system of moving loads shown in Fig. 4. What is the maximum shear: (a) at the left end? (b) at a point 10 feet from the left end?

SOLUTION.—(a) When W_1 is at the left end, all the loads are on the beam; when they move to the left, W_1 passes off. Then, if the loads move 8 ft. until W_2 is at the end, the increase in the shear is $\frac{8}{40} \times 44,000 = 8,800$ lb., and the decrease is

$$\frac{5,000 \times 40}{40} = 5,000 \text{ lb.}$$

The shear is therefore greater when W_2 is at the end. If the loads move 5 ft. more to the left until W_3 is at the end, the increase in the shear is $\frac{5}{40} \times 34,000 = 4,250$ lb., and the decrease is $10,000 \times \frac{40}{40} = 10,000$ lb. The shear is therefore greater when W_3 is at the end, and its value is

$$\frac{\Sigma Wx}{l} = (10,000 \times 40 + 10,000 \times 35 + 8,000 \times 30 + 8,000 \times 24 + 8,000 \times 18) \div 40 = 33,150 \text{ lb.} \quad \text{Ans.}$$

(b) When W_1 is at C , 10 ft. from the left end, W_2 is at the right end, and, since it will come on the span if the loads move to the left, it may be considered as a part of ΣW . Then, if the loads move 8 ft. to the left, the increase in shear is $\frac{8}{40} \times 49,000 = 9,800$ lb., and the decrease is 5,000 lb. If the loads move 5 ft. farther to the left, the increase in shear is

$$\frac{5}{40} \times 44,000 + \frac{2}{40} \times 5,000 = 5,750 \text{ lb.},$$

and the decrease is 10,000 lb. Therefore, the shear is a maximum when W_1 is at C , and its value is

$$\frac{\Sigma Wx}{l} - W_1 = (5,000 \times 38 + 10,000 \times 30 + 10,000 \times 25 + 8,000 \times 20 + 8,000 \times 14 + 8,000 \times 8) \div 40 - 5,000 = 21,900 \text{ lb.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. A beam 50 feet long supports the system of moving concentrated loads shown in Fig. 5. What is the maximum shear: (a) at the left end? (b) at a section 5 feet from the left end? (c) at a section 15 feet from the left end? (d) at the center of the beam?

$$\text{Ans. } \begin{cases} (a) 92,400 \text{ lb.} \\ (b) 78,900 \text{ lb.} \\ (c) 50,600 \text{ lb.} \\ (d) 26,400 \text{ lb.} \end{cases}$$

2. A beam 40 feet long supports the system of moving concentrated loads shown in Fig. 5. What is the maximum shear: (a) at the left end? (b) at the center?

$$\text{Ans. } \begin{cases} (a) 81,750 \text{ lb.} \\ (b) 23,000 \text{ lb.} \end{cases}$$

CONCENTRATED-LOAD MOMENTS AND SHEARS IN TRUSSES

MAXIMUM MOMENT AT A JOINT

17. Joint on the Loaded Chord.—In order to find the maximum moment at a joint of a truss, due to a system of moving concentrated loads, it is necessary first to find the position occupied by the loads for which the moment at the joint under consideration is a maximum. For this purpose, it will be well to consider the manner in which concentrated loads are transmitted to the panel points and abutments. Let W , Fig. 11, be any load moving over a bridge AB ; when it is at a floorbeam, it is wholly transmitted to one panel

point; when it is between two floorbeams, it is distributed between the two adjacent panel points. In the latter case, the amount that goes to each panel point may be found by resolving W into two components acting vertically through

N and M , whose values are, respectively, $W_N = W \times \frac{m}{p}$ and

$W_M = W \times \frac{n}{p}$. To find the reaction at either end, W_N and

W_M may be multiplied by their respective distances from the

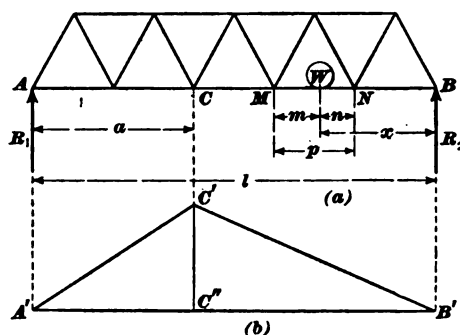


FIG. 11

other end, and the sum of the products divided by the length of the span; as, however, W is the resultant of W_N and W_M , the moment of W about any point (such as Wx about B) is the same as the sum of the moments of W_N and W_M about

the same point. The resultant moment of these components about B is $W_N(x - n) + W_M(x + m) = W \frac{m}{p} \times (x - n)$

+ $W \frac{n}{p} \times (x + m) = Wx$. The reactions and bending moments at the panel points can, therefore, be found without computing the panel loads.

The load in the end panel is transmitted partly to the first panel point of the truss and partly to the abutment: the former is supported by the truss; the latter goes directly to the abutment from the stringers, or, if there is an end floor-beam, this transmits it to the end joint of the truss. In either case, it simply increases the reaction, and causes no stresses in the members of the truss (except the vertical end post of a deck truss), and for this reason the half-panel load at the end was omitted in calculating moments and shears (*Stresses in Bridge Trusses*, Parts 1, 2, and 3). In the present

case, this portion of the load will be different for each different position of the loads, and it will be more convenient to compute the reaction for all the loads on the span, including those in the end panel, in exactly the same way as has already been done in the case of a simple beam. Then, *the maximum moment at any joint of the loaded chord of a truss is the same as the maximum moment at the corresponding point of a simple beam of the same span similarly loaded.* The broken line $A' C' B'$, Fig. 11 (b), is the influence line for the moment at C .

18. Moment on Unloaded Chord.—In the Pratt and other types of simple trusses, in which the joints of the unloaded chord are vertically opposite those of the loaded chord, the moment at any joint of the unloaded chord is the same as that at the opposite joint of the loaded chord. In the Warren and other types of trusses in which the joints of the unloaded chord are not vertically opposite those of the loaded chord, the calculation of moments at the joints of the unloaded chord requires special consideration.

19. Single Moving Load.—The variation in moment at a joint of the unloaded chord of a Warren truss can best

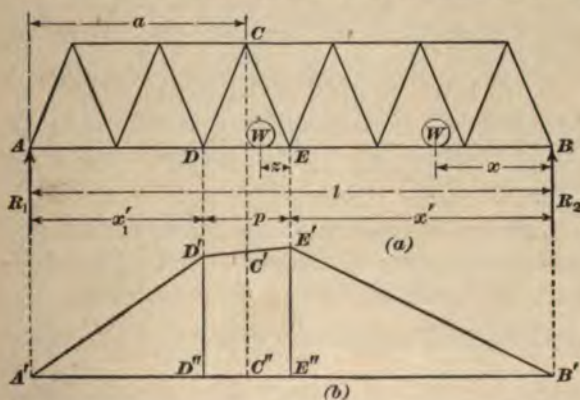


FIG. 12

be illustrated by considering the effect of a single moving load, such as W , Fig. 12 (a). As W moves from B to E ,

the moment at C is equal to $R_1 \times a$; if x is the distance of W from B when W is between B and E , R_1 is equal to $\frac{Wx}{l}$, and, as x increases uniformly, the moment at C , which

is $\frac{Wx}{l} \times a$, increases uniformly until W reaches E . The

variation in moment at C as W moves from B to E is represented by $B'E'$, Fig. 12 (*b*). When W is between E and D , a portion of it, W_D , is transmitted to D , and the remainder, W_E , to E ; then, the moment at C is equal to $R_1 \times a - W_D \times \frac{p}{2}$.

It has been shown that $R_1 \times a$ increases uniformly as W moves toward the left; if z represents the distance of W from E , then W_D is equal to $\frac{Wz}{p}$, and, as z increases uniformly, W_D , and therefore $W_D \times \frac{p}{2}$, must increase

uniformly; hence, the difference between $R_1 \times a$ and $W_D \times \frac{p}{2}$

must change at a uniform rate; that is, the influence line from E to D must be straight. The variation in moment at C as W moves from E to D is represented by $E'D'$, Fig. 12 (*b*). When W is at the left of D , it is more convenient to consider the right-hand reaction; if x_1 denotes the distance from A to W , the moment at C is equal to $\frac{Wx_1}{l} \times (l-a)$. As x_1 changes uniformly, the moment at C decreases uniformly as W moves from D to A . The variation in moment at C as W moves from D to A is represented by $D'A'$, Fig. 12 (*b*).

20. Influence Line for the Moment at C .—If x_1' represents the distance of D from A and x' the distance of E from B , the influence line for the moment at C may be drawn by laying off to scale $E'E''$ equal to $1 \times \frac{x'}{l} \times a$, and $D'D''$ equal to $1 \times \frac{x_1'}{l} \times (l-a)$, and connecting the points so found with each other and with B' and A' , respectively, forming the influence line $A'D'E'B'$. It may be shown, in

a similar manner, that the influence line between two consecutive joints of the loaded chord of a truss is always a straight line. In the present case, it is evident that the moment at C is a maximum when the single load is at E , and is represented by $C'E''$; in any case, the moment at a joint of the unloaded chord of the simple Warren truss, due to a single moving load, is a maximum when the load is at one of the panel points at the end of the panel opposite the joint.

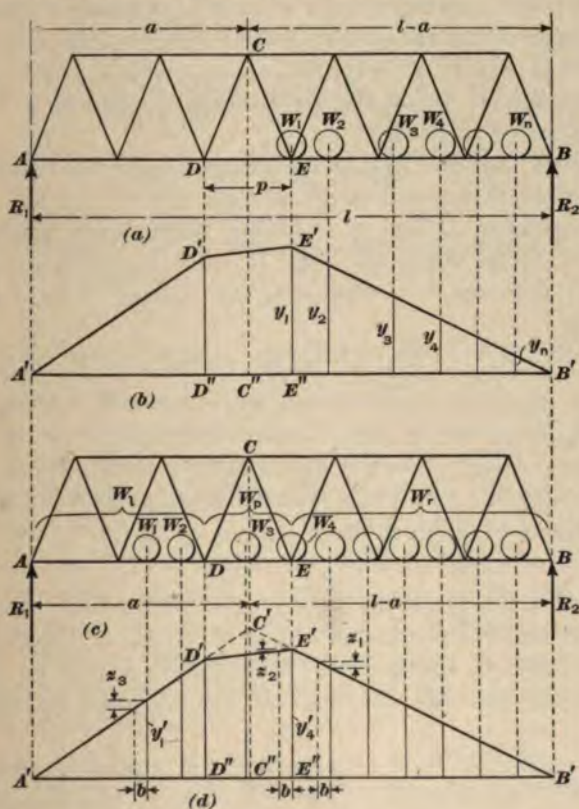


FIG. 13

21. Any Number of Loads.—Let AB , Fig. 13 (a), be a Warren truss over which a system of concentrated loads is moving, and let it be required to determine the position of

the loads when they cause maximum moment at C , at a distance a from the left end. Let $A'D'E'B'$, Fig. 13 (*b*), be the influence line for the moment at C . As the loads move on the span at B , and W_1 approaches E , the moment at C increases until W_1 reaches E ; if y_1, y_2 , etc. represent the ordinates to the influence line at W_1, W_2 , etc., the moment at C is equal to $W_1 y_1 + W_2 y_2 + \dots + W_n y_n$. Now, if the loads move a small distance to the left, W_1 will pass E ; y_2, y_3 , etc. will each increase by an amount that may be called z_1 ; and y_1 will decrease by an amount z_1 ; then, the change in moment is equal to $(W_2 + W_3 + \dots + W_n) z_1 - W_1 z_1$, and the moment is increased or decreased according as this is positive or negative.

Suppose that the moment increases until W_1 reaches E , and that in this position W_1 and W_2 are to the left of D ; W_3 is between D and E , and the other loads are to the right of E , as shown in Fig. 13 (*c*). If the loads move a small distance b to the left, the change in moment is equal to

$$(W_3 + W_4 + \dots + W_n) z_1 - [(W_2 + W_1) z_1 + (W_1 + W_2) z_1]$$

and the moment is increased or decreased according to whether the value of this change in moment is positive or negative. If the moment is a maximum when W_3 is at E , a portion of W_3 may be considered on the left and a portion on the right of E , so that

$$W_3 z_1 = W'_3 z_1 + W''_3 z_1 \quad (1)$$

letting W'_3 represent the sum of all the loads on the right of E , including a portion of the load at E ; W''_3 , the sum of all the loads in the panel DE , including a portion of the load at E ; and W_1 , the sum of all the loads to the left of D .

In Fig. 13 (*d*), if $A'D'$ and $B'E'$ are produced, they will meet at C' at distances a and $l - a$ from A' and B' , respectively. By similar triangles, the following values can be readily found:

$$z_1 = b \times \frac{C'C''}{C''B'} = b \times \frac{C'C''}{l - a}; \quad z_2 = b \times \frac{C'C''}{A'C''} = b \times \frac{C'C''}{a};$$

$$z_2 = b \times \frac{E' E'' - D' D''}{D'' E''} = \frac{b}{p} \left(C' C'' \frac{l - a - \frac{p}{2}}{l - a} - C' C'' \frac{a - \frac{p}{2}}{a} \right) \\ = b \times C' C'' \times \frac{\left(\frac{l}{2} - a \right)}{a(l - a)}$$

Substituting these values in equation (1),

$$W_r \times b \times \frac{C' C''}{l - a} = W_p \times b \times C' C'' \times \frac{\frac{l}{2} - a}{a(l - a)} + W_l \times b \times \frac{C' C''}{a};$$

whence
$$W_r = \frac{W_p l}{2a} - W_p + W_l \times \frac{l}{a} - W_l$$

and
$$\frac{W_r + W_p + W_l}{l} = \frac{\frac{W_p}{2} + W_l}{a}$$

or, denoting by ΣW the sum $W_r + W_p + W_l$ of all the loads on the span,

$$\frac{\Sigma W}{l} = \frac{\frac{W_p}{2} + W_l}{a}, \text{ and } W_l + \frac{W_p}{2} = \frac{a}{l} \times \Sigma W$$

From this, the following principle may be given for finding the maximum moment at a joint of the unloaded chord of a Warren truss: *The maximum moment due to a system of moving concentrated loads obtains when one of the loads is at the next panel point of the loaded chord to the right or left of the joint considered. The load at the panel joint must be such that, when considered part on one side and part on the other side of the panel point, the sum of one-half the loads in the panel opposite the given joint and the loads on the left of the panel, divided by the length of truss to the left of the joint, is equal to the average intensity of load on the span.*

22. Value of the Moment.—As the influence line, Fig. 13 (b), is straight between D and E , and as C is midway between D and E , the ordinate at C is equal to one-half the sum of the ordinates at D and E . It can be shown that, for any number of loads, the moment varies uniformly between panel points. The moment M_c at C is equal to

one-half the sum of the moments M_D and M_E at D and E , respectively; that is,

$$M_C = \frac{M_D + M_E}{2}$$

EXAMPLE.—What is the maximum moment due to the system of concentrated loads shown in Fig. 4, at the second joint C of the upper chord of a six-panel through Warren truss, assuming the span to have a length of 60 feet?

SOLUTION.—As the system of loads is shorter than the truss, it may be assumed that all the loads will be on when the moment is a maximum. Then, $a = 15$ ft., $l = 60$ ft., and

$$\frac{a}{l} \times \Sigma W = \frac{15}{60} \times 49,000 = 12,250 \text{ lb.}$$

Let D and E be the panel points to the left and right of C , respectively. When W_3 is at E , if all of W_4 is considered on the left of the point E , we have

$$W_l + \frac{W_p}{2} = \frac{15,000}{2} = 7,500 \text{ lb.}$$

As this is less than 12,250, the moment is not a maximum.

When W_3 is at E , if all of W_3 is considered on the right of the point E , we have

$$W_l + \frac{W_p}{2} = 5,000 + \frac{10,000}{2} = 10,000 \text{ lb.}$$

If all of W_3 is considered on the left of E ,

$$W_l + \frac{W_p}{2} = 5,000 + \frac{20,000}{2} = 15,000 \text{ lb.}$$

As one value is less and the other greater than 12,250, the moment is a maximum.

When W_4 is at E , W_3 is at D , and, if W_4 and W_3 are considered on the right of E and D , respectively, giving the smallest value of $W_l + \frac{W_p}{2}$, then,

$$W_l + \frac{W_p}{2} = 5,000 + \frac{20,000}{2} = 15,000 \text{ lb.}$$

As this is greater than 12,250, the moment is not a maximum. Therefore, the only position that causes a maximum moment at C , is when W_3 is at E . We have, then, for the reaction R_1 and the moments at the points D , E , and C , respectively,

$$R_1 = (5,000 \times 53 + 10,000 \times 45 + 10,000 \times 40 + 8,000 \times 35 + 8,000 \times 29 + 8,000 \times 23) \div 60 = 30,180 \text{ lb.}$$

$$M_D = 30,180 \times 10 - 5,000 \times 3 = 286,800 \text{ ft.-lb.};$$

$$M_E = 30,180 \times 20 - 5,000 \times 13 - 10,000 \times 5 = 488,600 \text{ ft.-lb.};$$

$$M_C = \frac{286,800 + 488,600}{2} = 387,700 \text{ ft.-lb.} \quad \text{Ans.}$$

EXAMPLE FOR PRACTICE

Let the eight-panel through Warren truss represented in Fig. 14 support the system of moving concentrated loads represented in Fig. 5.

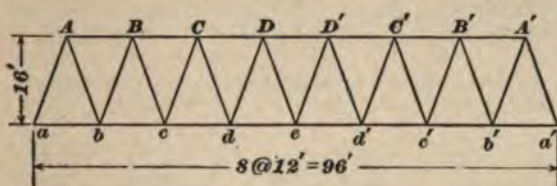


FIG. 14

What is the maximum moment: (a) at joint c ? (b) at joint e ? (c) at joint B ?

Ans. $\begin{cases} (a) & 2,032,500 \text{ ft.-lb.} \\ (b) & 2,680,000 \text{ ft.-lb.} \\ (c) & 1,589,400 \text{ ft.-lb.} \end{cases}$

MAXIMUM SHEAR IN A PANEL

23. Introduction.—In *Stresses in Bridge Trusses*, Part 1, it was explained that the shear due to a uniform load in any panel of a truss is uniform between the panel points; in the case of concentrated loads, they are transmitted to the trusses at the panel points, and, therefore, the shear at all points in any panel is the same. The position of the joints of the unloaded chord with respect to those of the loaded chord has no effect in the computation for shear.

24. Influence Line for Shear in a Panel.—Let AB , Fig. 15 (a), be a truss that supports a load of unity that moves from B to A , and let it be required to construct the influence line for the shear in panel CD . As the load moves from B to D , the shear is positive and equal to R_1 , and increases uniformly as the load moves toward D ; as the load moves from D to C , the shear decreases uniformly, and is equal to R_1 minus the portion that goes to C ; it becomes zero for some position of the load between D and C ; then it is negative and increases numerically until the load is at C , when the shear is $-R_2$. As the load moves from C to A , the shear changes from $-R_2$ to zero; that is, it increases algebraically, although its numerical value decreases from R_2

to zero. The broken line $B'D'C'A'$, Fig. 15 (b), is the influence line for shear in the panel CD , the portion $B'D'$ being parallel to the portion $C'A'$, as can be easily shown. The ordinates $D'D''$ and $C'C''$ are equal, respectively, to R_1 and R_2 , the former of these reactions being for the position D of the load, the latter for the position C . The point E where

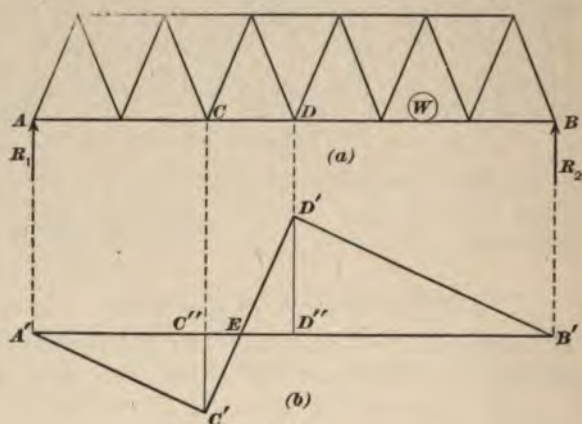


FIG. 15

$C'D'$ crosses $A'B'$ is where the load causes no shear in the panel CD . In this case, the portions of the load that go to C and D are equal to R_1 and R_2 , respectively, since the shear equals R_1 minus the portion going to C ; and, if the shear is equal to zero, then R_1 equals the portion going to C . It can be shown that the influence line for shear in any panel is straight between the panel points.

25. Any Number of Loads.—Let Fig. 16 (a) represent a truss over which a system of concentrated loads is moving, and let it be required to determine the position of the loads when they cause maximum positive shear in the panel CD . The influence line is shown in Fig. 16 (b). When the shear is a maximum, there will be few or no loads in the panel CD , and probably none to the left of C ; but, in order to generalize the discussion, it will be assumed that there are loads to the left of C as well as in the panel CD . Let W_1 , W_2 , and W_3

represent, respectively, the sum of the loads to the left of C , in the panel CD , and to the right of D ; and let ΣW be the sum of all the loads on the span. If, when the loads move a short distance b to the left, the changes in the ordinates to the influence line are represented by z_1 , z_2 , and z_3 , Fig. 16 (b),

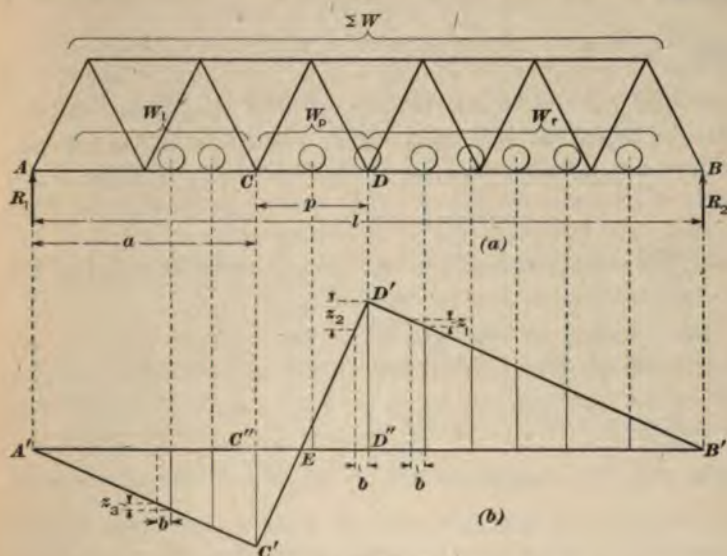


FIG. 16

the change in shear will be $W_l z_3 + W_r z_1 - W_p z_2$. The shear will be a maximum when a load is at D such that, if a portion of it is considered to the left and a portion to the right of D , the following equation obtains:

$$W_l z_3 + W_r z_1 = W_p z_2 \quad (1)$$

From similar triangles in Fig. 16 (b), the following values are obtained:

$$\begin{aligned} z_1 &= b \times \frac{D' D''}{D'' B'} = b \times \frac{D' D''}{l - a - p} \\ z_2 &= b \times \frac{C' C''}{A' C''} = b \times \frac{D' D''}{l - a - p} \\ z_3 &= b \times \frac{D' D'' + C' C''}{C'' D''} = \frac{b}{p} \left(D' D'' + a \times \frac{D' D''}{l - a - p} \right) \end{aligned}$$

Substituting these values in equation (1), multiplying by $\frac{l-a-p}{b \times D'D''}$, and reducing,

$$W_l + W_r + W_p = W_p \times \frac{l}{p};$$

whence

$$\frac{\Sigma W}{l} = \frac{W_p}{p}$$

and

$$W_p = \frac{p}{l} \times \Sigma W$$

Hence the following principle: *The maximum positive shear in any panel of a truss, due to a system of moving concentrated loads, obtains when one of the loads is at the joint to the right of the panel considered. The load at the panel point must be such that, if a portion of it is considered on each side of the joint, the average intensity of load in the panel is equal to the average intensity of load on the span.*

26. Value of the Shear.—The shear is equal to R , minus the portion of the loads in the panel that goes to the left of the panel, minus the sum of all the loads to the left of this joint. If Wx is the moment of any load about the right end, R , is equal to $\frac{\Sigma Wx}{l}$. If Wz is the moment of any load in the panel about the joint at the right of the panel, the portion that goes to the joint at the left of the panel is equal to $\frac{\Sigma Wz}{p}$. The shear is, therefore,

$$V = \frac{\Sigma Wx}{l} - \frac{\Sigma Wz}{p} - W_l$$

EXAMPLE.—The six-panel through Warren truss represented in Fig. 17 supports the system of moving concentrated loads shown in

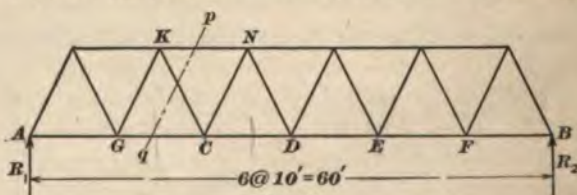


FIG. 17

Fig. 4. What is the maximum positive shear: (a) in the panel CD ? (b) in the panel DE ?

SOLUTION.—(a) When W_1 is at D , W_2 is at B , and may be considered as off the bridge; then,

$$W_p = \frac{10}{60} \times 41,000 = 6,833 \text{ lb.}$$

As W_1 is 5,000 lb., or less than 6,833 lb., the shear is not a maximum. When W_2 is at D , all the loads are on the span; then,

$$W_p = \frac{10}{60} \times 49,000 = 8,167 \text{ lb.}$$

$W_1 = 5,000$ lb., and $W_1 + W_2 = 15,000$ lb. As one is less and the other greater than W_p , the shear is a maximum when W_2 is at D . Then,

$$V = (5,000 \times 38 + 10,000 \times 30 + 10,000 \times 25 + 8,000 \times 20 + 8,000 \times 14 + 8,000 \times 8) \div 60 - \frac{5,000 \times 8}{10} = 13,930 \text{ lb. Ans.}$$

(b) When W_1 is at E , W_2 and W_3 are off the span; then,

$$W_p = \frac{10}{60} \times 33,000 = 5,500 \text{ lb.}$$

As this is greater than W_1 , the shear is not a maximum. When W_2 is at E , W_3 is off the span; then

$$W_p = \frac{10}{60} \times 41,000 = 6,833 \text{ lb.}$$

As this is greater than W_1 and less than $W_1 + W_2$, the shear is a maximum. Then,

$$V = \frac{5,000 \times 28 + 10,000 \times 20 + 10,000 \times 15 + 8,000 \times 10 + 8,000 \times 4}{60} - \frac{5,000 \times 8}{10} = 6,030 \text{ lb. Ans.}$$

EXAMPLE FOR PRACTICE

The eight-panel through Warren truss represented in Fig. 14 supports the system of moving concentrated loads represented in Fig. 5. What is the maximum positive shear: (a) in the panel $b c$? (b) in the panel $c d$? (c) in the panel $e d$?

$$\text{Ans. } \begin{cases} (a) 81,460 \text{ lb.} \\ (b) 63,340 \text{ lb.} \\ (c) 27,090 \text{ lb.} \end{cases}$$

MAXIMUM FLOORBEAM AND PANEL LOADS

27. Maximum Floorbeam Load.—The load that comes on a floorbeam is equal to the sum of the reactions on the stringers in the two adjacent panels. This is equal to twice the reaction from one stringer if the panels are equal and uniform loads are considered, but for concentrated loads the reactions will usually be different, and it is necessary to determine the position of the loads when they cause the maximum load on a floorbeam.

Let Fig. 18 (a) represent two adjacent panels of a floor system, of lengths a and a' , respectively, and let it be required to determine the maximum reaction at C as a system of concentrated loads moves from B to A . The stringers may be assumed to be simply supported at A , B , and C , so that the reactions at these points may be computed as usual. For example, the reaction at C due to W is $\frac{Wx}{a'}$. A load at C is supported directly by the floorbeam.

The line $B'C'A'$, Fig. 18 (b), is the influence line for the reaction at C , and, as this has the same form as the influence line for the moment at C in a beam of the length AB , it can

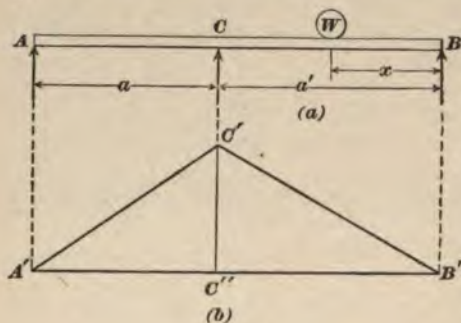


FIG. 18

be shown, in a manner similar to that employed in Art. 5, that the condition for maximum moment at the section C in a beam of length AB is also the condition for maximum reaction at C . That is: *The load on a floorbeam that comes from*

two adjacent panels is a maximum when such a load is at the floorbeam that, if considered part on each side, the average intensity of load on the panel to the left is equal to the average intensity of load on the panel to the right, and also equal to the average intensity of load on the two panels taken together.

In the ordinary case, the panel lengths a and a' are equal, and the principle just stated then reduces to the following:

The load on a floorbeam, when the panels on each side of it are equal, is a maximum when such a load is at the floorbeam that, if considered part on each side, the load in the panel to the left will be equal to the load in the panel to the right, and equal to one-half the sum of the loads in the two panels.

28. Maximum Panel Load.—The maximum panel load can be found from the maximum floorbeam load. In almost every case, the former will be equal to one-half the latter.

EXAMPLE.—What is the maximum panel load in a truss that supports the system of loads shown in Fig. 4, the panels being 15 feet long?

SOLUTION.—In determining maximum moments, it has been seen that the condition is usually satisfied when one of the heavier loads is at the section. In this case, then, it will be unnecessary to try W_1 at C . When W_2 is at C , the total load in both panels is 33,000 lb.; as one-half of this is greater than the sum of W_1 and W_4 , this position does not give a maximum.

When W_3 is at C , the total load is 41,000 lb. As one-half of 41,000 is greater than the sum of W_1 and W_4 , and less than the sum of W_1 , W_2 , and W_4 , this position fulfils the conditions for a maximum.

When W_4 is at C , the total load is equal to 44,000 lb. As one-half of 44,000 is greater than the sum of W_2 and W_3 , and less than the sum of W_2 , W_3 , and W_4 , this position fulfils the conditions for a maximum.

When W_2 is at C , the panel load is

$$\frac{5,000 \times 2 + 10,000 \times 10 + 8,000 \times 4 + 8,000 \times 10}{15} + 10,000 = 12,400 \text{ lb.}$$

When W_4 is at C , the panel load is

$$\frac{10,000 \times 5 + 10,000 \times 10 + 8,000 \times 3 + 8,000 \times 9}{15} + 8,000 = 12,200 \text{ lb.}$$

The panel load is, therefore, greatest when W_2 is at C ; it is equal to 12,400 lb. Ans.

EXAMPLES FOR PRACTICE

1. If the system of concentrated loads shown in Fig. 4 moves across a bridge, what is the maximum load that comes on a floorbeam supporting the adjacent ends of two panels 12 and 15 feet long, respectively?

Ans. 23,300 lb.

2. What is the maximum panel load on a truss that supports the system of loads shown in Fig. 5, when the length of panels is 20 feet?

Ans. 34,280 lb.

3. The adjacent ends of two spans 40 and 60 feet long, respectively, rest on the same pier. What is the maximum load on the pier when two systems of loads, each like that shown in Fig. 5, move over the spans, assuming the first (left) load of the second system to be 7 feet to the right of the last load of the first system?

Ans. 145,750 lb.

STRESSES DUE TO CONCENTRATED LOADS

PARALLEL-CHORD TRUSSES

SINGLE-SYSTEM TRUSSES

29. From the preceding principles, the live-load stresses in the members of any parallel-chord truss with a single system of web, and in the members of Warren trusses with secondary verticals, can be readily determined. As the maximum live-load stresses are usually desired, it is necessary to find the maximum live-load moments or shears, as explained in the preceding pages.

For example, to find the maximum live-load stress in the diagonal KC of the truss AB , Fig. 17, the shear in the panel GC is considered. The live-load tension in KC is greatest when the positive live-load shear in the panel GC (or on the plane of section pq) is a maximum; the live-load compression is greatest when the negative live-load shear in the panel GC is a maximum. As the maximum positive live-load shear in the panel EF is equal to the maximum negative live-load shear in the panel GC , the maximum live-load tension and compression in the diagonal KC are equal, respectively, to the product of $\csc H$ and the maximum positive shears in the panels GC and EF , found as explained in Art. 25 for the system of loads under consideration.

The live-load stress in the chord member KN , Fig. 17, is equal to the live-load moment at C divided by the height of the truss, and is greatest when the moment at C is a maximum. The maximum moment at C for the system of loads under consideration can be found as explained in Art. 17.

Similarly, the stress in GC can be found from the moment at K found as explained in Arts. 21 and 22.

In the following pages, whenever stresses are mentioned, live-load stresses are meant, unless dead-load stresses are specified.

The methods of finding the stresses in some of the members of the Baltimore truss and in the members of multiple-system trusses will now be discussed.

THE BALTIMORE TRUSS

30. In the Baltimore truss, Fig. 19 (*a*), the maximum stresses in the upper-chord members can be found from the maximum moments at the opposite joints; the maximum stresses in the lower halves of the main diagonals can be found from the maximum shears in the respective panels; and the maximum stresses in the subverticals and substruts can be found from the maximum panel loads. There remains to be considered the maximum stresses in the lower-chord members, in the upper halves of the main diagonals, in the counters, and in the main verticals.

31. Lower Chord.—The maximum stress in the end panel of the lower chord is equal to the horizontal component of the maximum stress in the lower half of the end post, which is found from the maximum shear in the end panel. The maximum stress in any other part of the lower chord, such as GD , Fig. 19 (*a*), is found from the moment at the center of moments in the upper chord, in this case joint E . The plane q that cuts GD and two other members is to the right of the panel load at G , while the center of moments is to the left of this panel load; the moment of the load at G is then opposite in sign to that of the loads to the left of C . On this account, the influence line for the moment at E is different from the influence lines that have already been discussed, being composed of three lines $B, D_1, D_1 G_1$, and $G_1 A_1$, Fig. 19 (*b*). Now, if W_1 denotes the sum of the loads in the length AG , W_2 the sum of the loads in the

length GD (the panel cut by the plane q), and ΣW the total load on the span AB , it may be shown, in a manner similar to that employed in Art. 21, that *the moment at E is a maxi-*

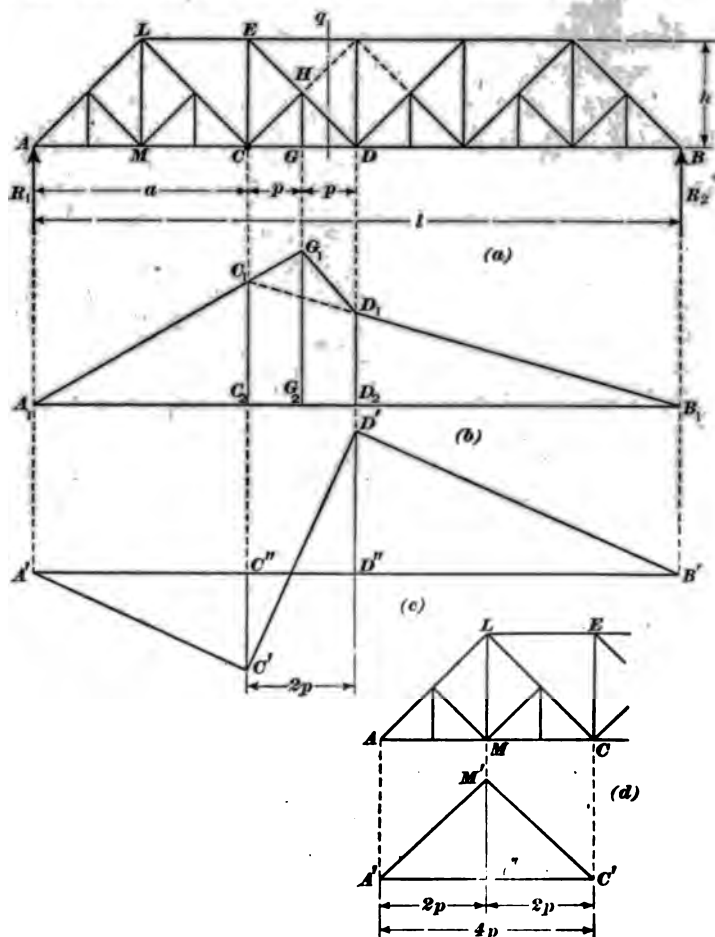


FIG. 19

um when such a load is at G that, if considered part on one side and part on the other, the difference between the loads to the left of G and those in panel GD , divided by the length of the truss to the left of the center of moments is equal to the average

intensity of the load on the span. That is, the stress in the lower chord is a maximum when

$$\frac{W_i - W_p}{a} = \frac{\Sigma W}{l}$$

or when

$$W_i - W_p = \frac{a}{l} \times \Sigma W$$

32. Upper Half of Main Diagonals.—The influence line for the vertical component of the stress in the upper half of a main diagonal, such as EH , Fig. 19 (*a*), may be drawn by computing the vertical component of the stress due to a load of unity that moves over the span. This line is composed of the three straight lines $B'D'$, $D'C'$, and $C'A'$, Fig. 19 (*c*); this is exactly the same as the influence line for shear in the panel CD of a Pratt truss having the same length and half as many panels as the truss represented in Fig. 19 (*a*). It can be shown (see Art. 25) that *the stress in EH is a maximum when there is such a load at D that, if considered part on each side, the average intensity of load in the double panel CD is equal to the average intensity of load on the span.* That is,

$$\frac{W_{2p}}{2p} = \frac{W}{l};$$

whence

$$W_{2p} = \frac{2p}{l} \times \Sigma W$$

The vertical component of the stress in EH can be found, as usual, by subtracting from the shear in panel CG one-half the panel load at G , both being computed when the loads are in a position that satisfies the condition just stated; the result is the same as though the shear were computed in the panel CD of the Pratt truss already mentioned.

33. Intermediate Verticals.—The stress in any intermediate vertical, such as EC , Fig. 19 (*a*), is a maximum when the stress in the adjacent diagonal is a maximum, and is equal to the vertical component of the maximum stress in that diagonal.

34. Hip Vertical.—The influence line for the stress in LM is represented in Fig. 19 (*d*), and has the same form as

the influence line for the moment at the center of a beam having a span equal to four panel lengths. Therefore, *the stress in the hip vertical is a maximum when such a load is at its foot that, if considered part on each side, the average intensity of the load in the two panels to the left is equal to the average intensity of the load in the two panels to the right.* It can be shown that the stress in LM is equal to the maximum panel load for a truss having a panel length equal to $2p$.

35. Counters.—When the upper half of a counter is in action, the lower half of the main diagonal in the same panel is out of action, and the stress in the counter may be found, as usual, from the shear. The vertical component of the stress in the lower half of a counter, such as CH , Fig. 19 (*a*), is equal to the maximum negative shear in the panel CD of a Pratt truss of the same span and half the number of panels.

EXAMPLE.—The Baltimore truss shown in Fig. 20 is one truss of a bridge that supports the system of concentrated loads followed by a uniform load represented in Fig. 21. What is the maximum stress: (*a*) in the upper-chord member CE ? (*b*) in the lower-chord member de ? (*c*) in the diagonal CD ? (*d*) in the vertical Cc ? (*e*) in the counter eF ? It is assumed that one-half the load is carried by one truss.

SOLUTION.—(*a*) The stress in CE is equal to the moment at e divided by 30. It can be found, by trial, that the first position that satisfies the condition for a maximum moment is when W_9 is at e . In this case, $\frac{a}{l} = \frac{4}{12} = \frac{1}{3}$. The numerical work can be arranged conveniently in tabular form as follows:

Position of Load	ΣW Pounds	$\frac{a}{l} \times \Sigma W$ Pounds	W_L Pounds
W_9 at e	748,000	249,300	Between 222,000 and 244,000
W_9 at e	768,000	256,000	Between 244,000 and 266,000
W_{10} at e	800,000	266,700	Between 266,000 and 284,000
W_{11} at e	814,000	271,300	Between 266,000 and 306,000
W_{12} at e	794,000	264,700	Between 266,000 and 306,000

As the values of $\frac{a}{l} \times \Sigma W$ for W_9 , W_{10} , and W_{11} lie between the extreme values of W_L , respectively, these three positions satisfy the

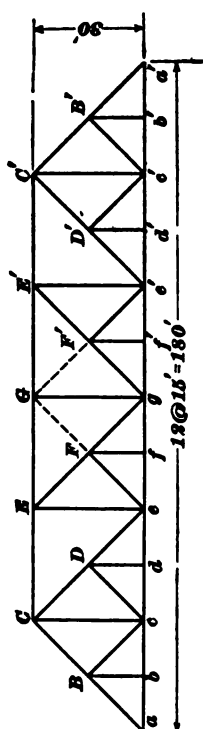


FIG. 20

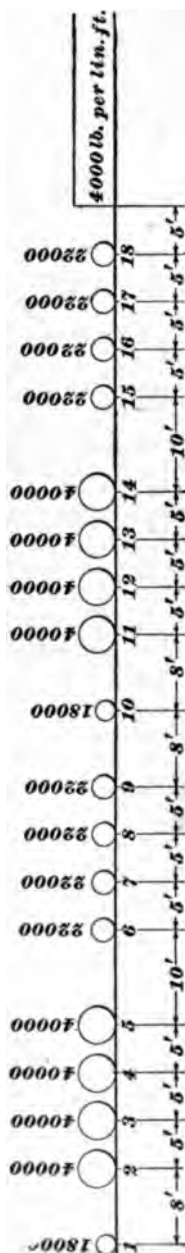
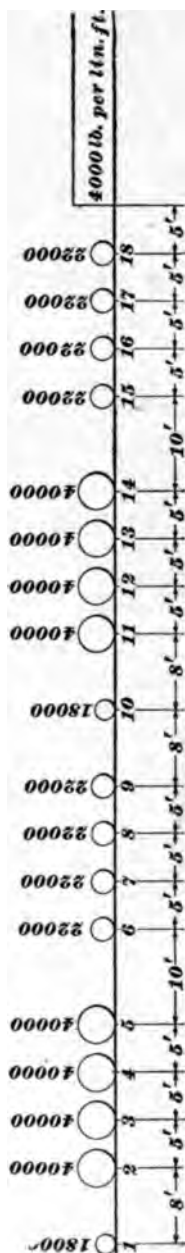
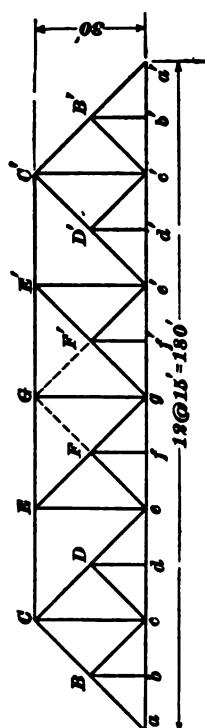


FIG. 21



condition for a maximum, and it is necessary to compute the moment for each position to find the greatest.

When W_5 is at e , $R_1 = 385,300$ lb., and the moment at e is
 $385,300 \times 60 - (18,000 \times 48 + 160,000 \times 32.5 + 88,000 \times 7.5)$
 $= 16,394,000$ ft.-lb.

When W_{10} is at e , $R_1 = 420,144$ lb., and the moment at e is
 $420,144 \times 60 - (18,000 \times 56 + 160,000 \times 40.5 + 88,000 \times 15.5)$
 $= 16,356,640$ ft.-lb.

When W_{11} is at e , $R_1 = 438,011$ lb., and the moment at e is
 $438,011 \times 60 - (160,000 \times 48.5 + 88,000 \times 23.5 + 18,000 \times 8)$
 $= 16,308,660$ ft.-lb.

The greatest of these moments occurs when W_5 is at e . Then, the stress in CE is

$$\frac{16,394,000}{2 \times 30} = 273,200 \text{ lb., compression. Ans.}$$

(b) The stress in de is equal to the moment at C divided by 30. It will be found, by trial, that the conditions for maximum moment at C are satisfied when W_5 , W_6 , W_{10} , and W_{13} , respectively, are at d . The results of the calculations are as follows:

Position of Load	ΣW Pounds	$\frac{a}{l} \times \Sigma W$ Pounds	$W_l - W_p$ Pounds
W_5 at d	708,000	118,000	Between 18,000 and 76,000
W_6 at d	728,000	121,300	Between 76,000 and 134,000
W_8 at d	768,000	128,000	Between 112,000 and 134,000
W_7 at d	788,000	131,300	Between 134,000 and 178,000
W_9 at d	810,000	135,000	Between 186,000 and 230,000
W_{10} at d	802,000	133,700	Between 110,000 and 146,000
W_{11} at a	754,000	125,700	Between 26,000 and 66,000
W_{12} at d	734,000	122,300	Between 26,000 and 106,000
W_{13} at d	754,000	125,700	Between 106,000 and 164,000
W_{14} at d	752,000	125,300	Between 142,000 and 200,000

Since the value of $\frac{a}{l} \times \Sigma W$ lies between the extreme values of $W_l - W_p$ when W_5 , W_6 , W_{10} , and W_{13} , respectively, are at d , the conditions for maximum moment at C are fulfilled for each of these positions, and it is necessary to calculate the moment for each position to find the greatest moment.

The moment at C is equal to R_1 multiplied by 30, minus the moment of all the loads to the left of e about C , plus the moment of the panel load at d about C , as explained in *Stresses in Bridge Trusses*, Part 2.

When W_6 is at d , R_1 is 343,744 lb., and the panel load at d is 87,333 lb. Then, the moment at C is

$$343,744 \times 30 - 18,000 \times 8 + 87,333 \times 15 = 11,478,320 \text{ ft.-lb.}$$

When W_8 is at d , R_1 is 385,300 lb., and the panel load at d is 57,333 lb. Then, the moment at C is

$$385,300 \times 30 - [(18,000 \times 18) + (40,000 \times 10) + (40,000 \times 5)] + 57,333 \times 15 = 11,495,000 \text{ ft.-lb.}$$

When W_{10} is at d , R_1 is 429,544 lb., and the panel load at d is 55,200 lb. Then, the moment at C is

$$429,544 \times 30 - (120,000 \times 23 + 22,000 \times 8 + 22,000 \times 3) + 55,200 \times 15 = 10,712,320 \text{ ft.-lb.}$$

When W_{12} is at d , R_1 is 386,011 lb., and the panel load at d is 106,667 lb. Then, the moment at C is

$$386,011 \times 30 - [(88,000 \times 18.5) + (18,000 \times 3)] + 106,667 \times 15 = 11,498,330 \text{ ft.-lb.}$$

The last value found is the greatest, so the moment at C is greatest when W_{12} is at d . The maximum stress in de is, then,

$$\frac{11,498,330}{2 \times 30} = 191,640 \text{ lb., tension. Ans.}$$

(c) The stress in CD is a maximum when the average intensity of load in the double panel ce is equal to the average intensity of load on the span. In this case, $\frac{2p}{l} = \frac{1}{6}$.

When W_3 is at e , $\Sigma W = 628,000$ lb.; $\frac{2p}{l} \times \Sigma W = 104,700$ lb.; W_{2p} is between 58,000 lb. and 98,000 lb.

When W_4 is at e , $\Sigma W = 648,000$ lb.; $\frac{2p}{l} \times \Sigma W = 108,000$ lb.; W_{2p} is between 98,000 lb. and 138,000 lb.

Then, the stress is a maximum when W_4 is at e ; in this position, $R_1 = 267,300$ lb.

The loads at c and d , due to the loads in the panels cd and de , are 3,600 lb. and 54,400 lb., respectively. Then, the vertical component of the stress in CD is

$$\frac{267,300 - 3,600 - \frac{54,400}{2}}{2} = 118,250 \text{ lb.}$$

and the stress in CD is

$$118,250 \times \frac{\sqrt{15^2 + 15^2}}{15} = 167,200 \text{ lb., tension. Ans.}$$

(d) The stress in Cc is a maximum when the average intensity of load in the length ac is equal to the average intensity of load in the length ce . There will probably be more loads in the length ac when the second set of heavy loads is in that position, so it is unnecessary to

try W_2 , W_3 , etc. at c . The results of the calculations for several positions are as follows:

Position of Load	ΣW Pounds	$\frac{a}{l} \times \Sigma W$ Pounds	Wl Pounds
W_{11} at c	266,000	133,000	Between 84,000 and 124,000
W_{12} at c	266,000	133,000	Between 102,000 and 142,000
W_{13} at c	266,000	133,000	Between 120,000 and 160,000
W_{14} at c	266,000	133,000	Between 138,000 and 178,000

The condition for maximum stress in Cc is satisfied when W_{12} is at c and when W_{14} is at c .

When W_{12} is at c , the stress in Cc is 82,050 lb.

When W_{14} is at c , the stress in Cc is 82,750 lb., tension. Ans.

(e) The maximum stress in eF is the same as that in $e'F'$ when the right end of the truss is loaded; this occurs when the average intensity of load in the double panel $g'e'$ is equal to the average intensity of load on the span.

When W_2 is at e' , $\Sigma W = 324,000$ lb.; $\frac{2p}{l} \times \Sigma W = 54,000$ lb.; W_{12} is between 18,000 lb. and 58,000 lb.

When W_3 is at e' , $\Sigma W = 364,000$ lb.; $\frac{2p}{l} \times \Sigma W = 60,700$ lb.; W_{12} is between 58,000 lb. and 98,000 lb.

Both positions fulfil the conditions for maximum stress.

When W_2 is at e' , R_1 for one truss is 34,500 lb., and the load at f' is 4,800 lb. The vertical component of the stress in $e'F'$ is

$$34,500 - 4,800 + \frac{4,800}{2} = 32,100 \text{ lb.}$$

When W_3 is at e' , R_1 for one truss is 39,450 lb., and the load at f' is 14,450 lb. The vertical component of the stress in $e'F'$ is

$$39,450 - 14,450 + \frac{14,450}{2} = 32,200 \text{ lb.}$$

Then, the stress in $e'F'$ is

$$32,200 \times \frac{\sqrt{15^2 + 15^2}}{15} = 45,500 \text{ lb., tension. Ans.}$$

MULTIPLE-SYSTEM TRUSSES

36. Influence Lines.—Fig. 22, (b) and (c), shows the influence lines for the stresses in the chord member FH , and the web member HE , respectively, of the Whipple truss AB shown in Fig. 22 (a); they are typical influence

lines for the stresses in the members of multiple-system trusses.

In drawing the influence line for the stress in a member of a multiple-system truss, the truss is separated into simple systems, as explained in *Stresses in Bridge Trusses, Part 2*,

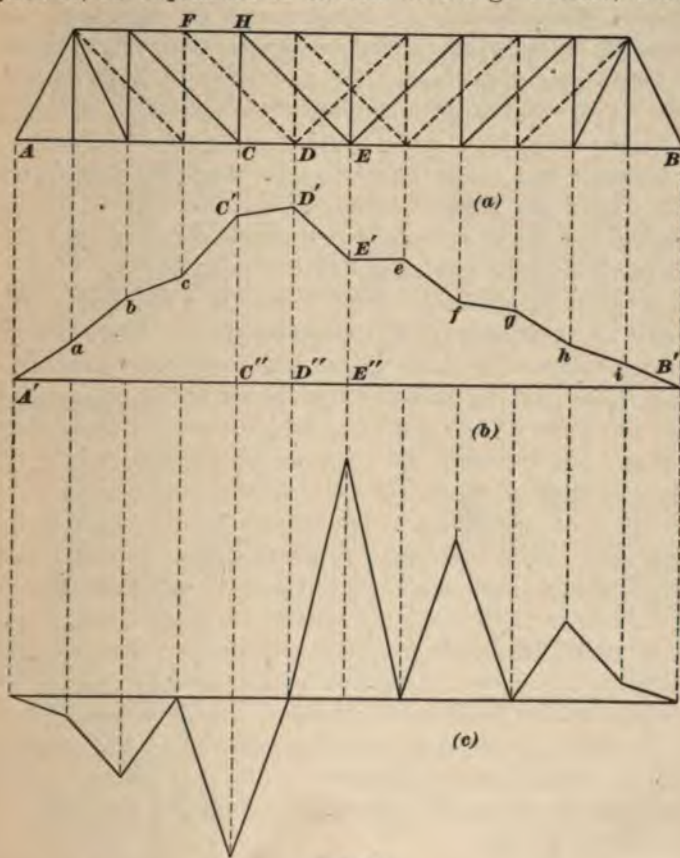


FIG. 22

and a load of unity is placed successively at the different panel points. The stress in the member under consideration is calculated for each position of the load, and is plotted to scale vertically under the panel point as an ordinate (such as C'' C' , D'' D' , E'' E' , etc.), to a horizontal line $A'B'$. The

line $A'abcC'D'E'$, etc. connecting the points so located is the influence line for the stress in the member. As these lines are very irregular, the principles that govern the conditions for the maximum stress in a member are very complicated and not adapted to practical use. If desired, however, the stress in a member can be found, by the method of influence lines, by plotting to scale the influence line for the member, laying off the series of loads successively in several positions, one of which will probably cause the maximum stress, and scaling the ordinate to the influence line under each load for each position of the loads. Then the algebraic sum of the products of all the loads on the span, and their respective ordinates for any position, will equal the stress in the member.

In order to find the maximum stress in a member, it will be well to determine first the positions the loads occupy when the moment or shear is a maximum at the section of a simple beam (having the same span as the truss) corresponding to the point of the influence line where the ordinate is greatest. For example, for the maximum stress in HE , the loads may first be placed in the position they occupy when the shear is a maximum at the section E of a beam of the length AB . With the loads in this position, the stress may be calculated as just described; then, with the loads moved successively in each direction until other loads come at the desired point, the stress may be calculated for several positions until it is seen that the stress decreases by moving the loads any farther in either direction. The maximum stress will usually be found after two or three trials. This method is simple in application and gives results that are sufficiently accurate if the influence line is constructed and the ordinates scaled very carefully.

37. Panel Concentrations.—The method of concentration by panels consists in determining, in the same way as in the preceding article, the probable position the loads occupy when the stress in a member is a maximum, and, when the loads are in that position, calculating the amount that goes

to each panel point of the truss. With these panel loads, the stresses may be found in the same way as in *Stresses in Bridge Trusses*, Part 2. By repeating this process for two or three positions, the maximum stress will be found. In finding stresses in web members, it is only necessary to compute the panel loads for the system to which the member belongs.

EXAMPLE.—Let the double Warren truss represented in Fig. 23 support one-half of the system of moving concentrated loads represented in Fig. 21. When W_{11} is at the center of the span, what are the panel concentrations?

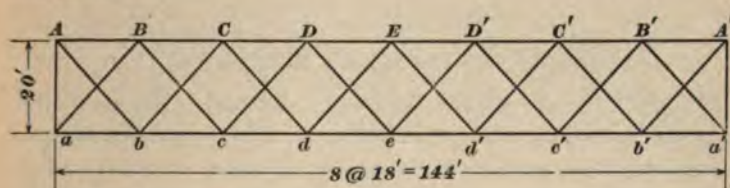


FIG. 23

SOLUTION.—

PANEL POINT	PANEL LOAD, IN POUNDS
a	$\frac{40,000 \times 2 + 18,000 \times 10}{2 \times 18} = 7,200$
b	$\frac{40,000 \times (5 + 10 + 15 + 16) + 18,000 \times 8}{2 \times 18} = 55,100$
c	$\frac{22,000 \times (3 + 8 + 13) + 40,000 \times (3 + 8 + 13)}{2 \times 18} = 41,300$
d	$\frac{18,000 \times 8 + 22,000 \times (16 + 15 + 10 + 5)}{2 \times 18} = 32,100$
e	$\frac{22,000 \times 2 + 18,000 \times 10 + 40,000 \times (18 + 13 + 8 + 3)}{2 \times 18} = 52,900$
d'	$\frac{40,000 \times (5 + 10 + 15) + 22,000 \times (1 + 6 + 11)}{2 \times 18} = 44,300$
c'	$\frac{22,000 \times (7 + 12 + 17 + 14) + 36,000 \times 4.5}{2 \times 18} = 35,100$
b'	$\frac{22,000 \times 4 + 36,000 \times 13.5 + 72,000 \times 9}{2 \times 18} = 33,900$
a'	$\frac{72,000 \times 9}{2 \times 18} = 18,000$

EXAMPLE FOR PRACTICE

If the double Warren truss represented in Fig. 23 supports one-half of the system of moving concentrated loads shown in Fig. 21, what are the panel loads when W_{12} is at c ?

Ans.	PANEL POINT	PANEL LOAD, IN POUNDS	PANEL POINT	PANEL LOAD, IN POUNDS
	a	17,500	d'	37,400
	b	59,300	c'	34,100
	c	37,900	b'	35,100
	d	30,400	a'	18,000
	e	60,300		

INCLINED-CHORD TRUSSES

38. By means of the principles already discussed, the stresses in inclined-chord trusses can be found in all the members that depend simply on the moment or shear. For example, the stresses in the chord members can be found directly from the moments; the stress in the end post, from the shear in the end panel; and the stress in the hip vertical, from the maximum panel load. The stresses in the intermediate verticals and diagonals require separate consideration.

THE CURVED-CHORD TRUSS

39. Maximum Stress in a Diagonal.—Let it be required to determine the position of a system of moving concentrated loads when the stress in the diagonal ED of the truss represented in Fig. 24 (a) is a maximum. The influence line for the stress in ED is composed of the three straight lines $B'D'$, $D'C'$, and $C'A'$, and is drawn by computing the stress in ED due to a load of unity that moves over the span; the ordinate $D''D'$ represents the stress in ED when the load is at D ; etc. Using the same method of proof as in previous cases, it can be shown that the stress in ED is a maximum when $\frac{W_r}{a'} + \frac{W_l}{a} \times \frac{CF}{FD} = \frac{W_r}{FD}$, W_r , W_l , and W_t , representing, as usual, the sum of all the loads to the right of D , to the left of C , and in the panel CD , respectively, there being a load at D . Under ordinary conditions, there

will be few loads in the panel CD and no loads to the left of C . Making W_i equal to zero, the preceding equation reduces to

$$\frac{W_r}{a'} = \frac{W_p}{FD};$$

or

$$\frac{\sum W - W_p}{a'} = \frac{W_p}{FD},$$

whence

$$W_p = \frac{FD}{a' + FD} \sum W = \frac{FD}{FB} \sum W,$$

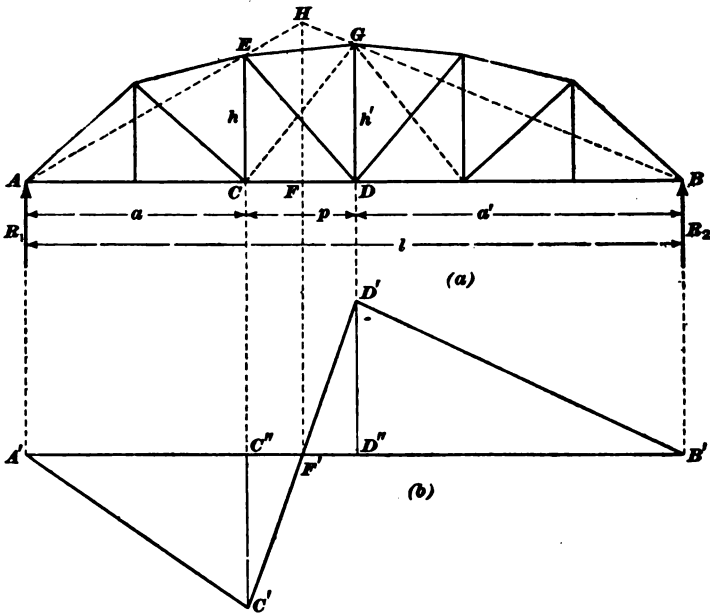


FIG. 24

which is a convenient formula to use when the point F has been located.

To locate the point F , consider that, when a load is at F , the stress in ED due to that load is zero; then the stress in CD is equal to the horizontal component of the stress in EG .

The stress in CD is $1 \times \frac{FB}{l} \times \frac{a}{h}$; the horizontal component

of the stress in EG is $1 \times \frac{AF}{l} \times \frac{a'}{h'}$; then,

$$\frac{FB}{l} \times \frac{a}{h} = \frac{AF}{l} \times \frac{a'}{h'}$$

whence

$$\frac{AF}{FB} = \frac{a h'}{a' h}$$

that is,

$$\frac{l - FB}{FB} = \frac{a h'}{a' h}$$

and

$$FB = l \times \frac{a' h}{a h' + a' h}$$

Also,

$$FD = FB - a'$$

For all ordinary purposes, it is sufficiently accurate to locate the point F graphically, laying out the truss carefully to scale and drawing the lines AE and BG through the joints A and E , B and G , respectively. Then F lies vertically under their intersection H . From similar triangles, in Fig. 24 (a),

$$\frac{FH}{AF} = \frac{h}{a}, \text{ and } \frac{FH}{FB} = \frac{h'}{a'}$$

whence, as before,

$$\frac{AF}{FB} = \frac{a h'}{a' h}$$

EXAMPLE.—Let the eight-panel truss represented in Fig. 25 support one-half of the system of moving concentrated loads represented in

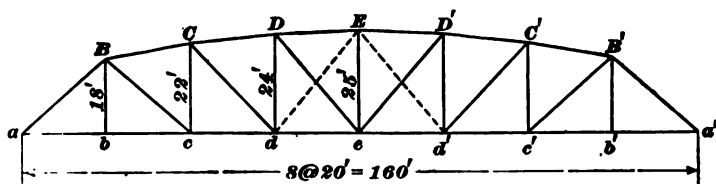


FIG. 25

Fig. 21, and let it be required to find the maximum stress in the diagonal Cd .

SOLUTION.—In this truss, Fa' (the point F is not shown) corresponds to FB in Fig. 24. Also, $l = 160$, $h = 22$, $h' = 24$, $a = 40$, $a' = 100$. Therefore,

$$Fa' = 160 \times \frac{100 \times 22}{40 \times 24 + 100 \times 22} = 111.39 \text{ ft.}$$

and $Fd = 11.39$ ft. Then, $\frac{Fd}{Fa'} = \frac{11.39}{111.39} = .1023$ or .1, nearly.

When W_2 is at d , $\Sigma W = 532,000$ lb.; $\frac{Fd}{Fa'} \times \Sigma W = 53,200$ lb.; W_2 is between 18,000 lb. and 58,000 lb. Therefore, the stress is a maximum

when W_2 is at d . R_1 for one truss is 95,225 lb.; the panel load at c is 3,600 lb. The shear in the panel cd is $95,225 - 3,600 = 91,625$ lb. The vertical component in CD is

$$\frac{95,225 \times 60 - 3,600 \times 20}{24} \times \frac{2}{20} = 23,500 \text{ lb.}$$

Then, the stress in Cd is

$$(91,625 - 23,500) \times \csc Cdc = 68,125 \times \frac{\sqrt{22^2 + 20^2}}{22} = 92,000 \text{ lb., tension.}$$

Ans.

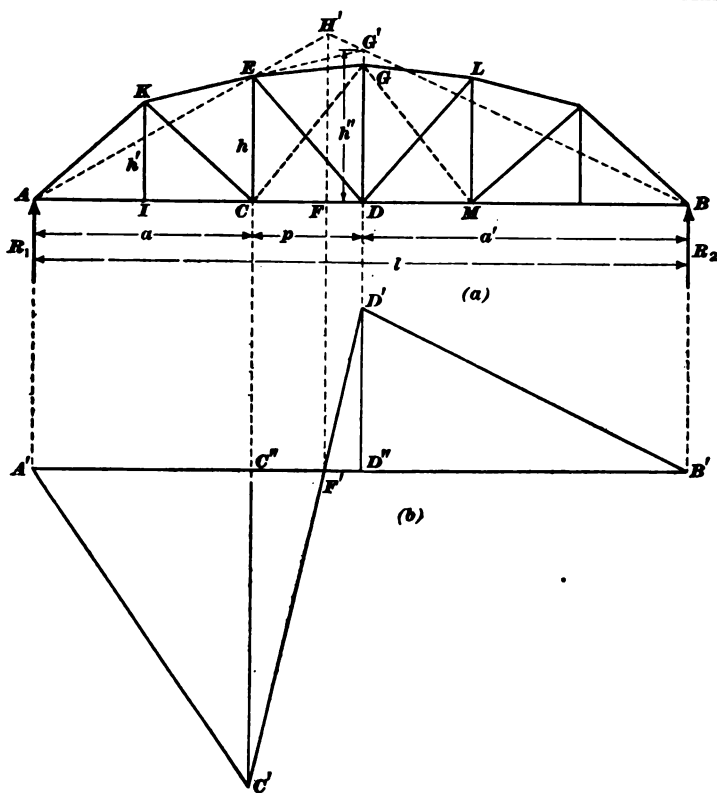


FIG. 26

40. Maximum Stress in a Vertical.—Let it be required to determine the position of a system of moving concentrated loads when the stress in the vertical EC of the truss represented in Fig. 26 is a maximum. The influence line for the stress in EC , Fig. 26, is similar to that for the

diagonal ED , and it may be shown that the same conditions determine the position for the maximum stress in the vertical as in the diagonal, the only difference being in the location of the point F . Then, as in the case of the diagonal,

$$\frac{W_r}{a'} + \frac{W_l}{a} \times \frac{CF}{FD} = \frac{W_p}{FD}$$

and, when $W_l = 0$,

$$W_p = \frac{FD}{FB} \times \Sigma W$$

To locate the point F , consider that, when a load is at F , the stress in EC due to that load is equal to zero, and the vertical component of the stress in KE is equal to the shear on a plane of section that cuts KE , EC , and CD ; that is, to the shear in the panel CD .

The shear is equal to $1 \times \frac{FB}{l} - 1 \times \frac{FD}{p}$, and the vertical component in KE is equal to

$$1 \times \frac{FB}{l} \times \frac{a}{h} \times \frac{h-h'}{p}$$

$$\text{Then,} \quad \frac{FB}{l} - \frac{FD}{p} = \frac{FB}{l} \times \frac{a}{h} \times \frac{h-h'}{p};$$

whence, substituting for FD its equivalent $FB - a'$, and solving for FB ,

$$FB = l \times \frac{a'h}{h(a+a') + a(h-h')}$$

Now, if KE is produced to meet the vertical GD produced at G' , $G'D - CE = CE - KI$, or $h'' - h = h - h'$. Substituting $h'' - h$ for $h - h'$ in the foregoing expression, and reducing, the following equation is obtained:

$$FB = l \times \frac{a'h}{ah'' + a'h}$$

which is also the value of FB for the diagonal ED in a truss having the height h'' at D . Hence, F may be found by drawing the lines AE and BG' to their intersection H' and drawing the vertical $H'E$, as in the case of the diagonal.

EXAMPLE.—What is the maximum stress in the vertical Cc of the truss represented in Fig. 25, due to the system of loads represented in Fig. 21?

SOLUTION.—In this case, $h = 22$ ft. and $h' = 18$ ft.; therefore, $h'' = 26$ ft.

$$Fa' = 160 \times \frac{100 \times 22}{40 \times 26 + 100 \times 22} = 108.64 \text{ ft.}, Fd = 8.64 \text{ ft.};$$

$$\frac{Fd}{Fa'} = \frac{8.64}{108.64} = .08, \text{ nearly.}$$

When W_2 is at d , $\Sigma W = 532,000$ lb.; $\frac{FD}{FB} \times \Sigma W = 42,560$ lb.; and

W_2 is between 18,000 lb. and 58,000 lb. Then, the stress is a maximum when W_2 is at d . $R_1 = 95,225$ lb., and the load at $c = 3,600$ lb. BC and cd intersect 70 ft. to the left of a . Then, the stress in Cc is

$$\frac{95,225 \times 70 - 3,600 \times 110}{100} = 57,000 \text{ lb., compression. Ans.}$$

41. Minimum Stress in a Vertical.—The minimum stresses in all the verticals not adjacent to panels containing counters can be found from the principles already explained. The minimum stress in any other vertical is tension; it occurs when the two diagonals that meet the vertical at one end are in action, and is equal to the algebraic sum of the vertical components of the stresses in the chord members that meet the vertical at the other end. The minimum stress (so called) in the center vertical obtains when the moment at the center of the span is a maximum. The minimum stress in the other verticals can best be found by trial. For example, for the minimum stress in EC (LM), Fig. 26 (a), the loads should extend over as great a portion of the right end of the truss as possible without throwing the counter GM out of action. This position can be ascertained by trying several loads successively at M until it is found that any further movement of the loads causes GM to go out of action. The last position of the loads before GM goes out of action is the position for which the stress is a minimum.

THE PETIT TRUSS

42. As in the case of the Baltimore truss, the stresses in the members of the upper chord and in the end panels of the lower chord of a Petit truss can be found from the bending moments at the opposite joints; the loading causing the maximum stresses in the remaining members of the

lower chord can be found by means of the formula

$$W_l - W_r = \frac{a}{l} \times \Sigma W$$

explained in Art. 31; the stresses in the subverticals and short diagonals can be found from the panel load; and the stress in the hip vertical, from the panel load for a truss having panels twice the length of the Petit truss. There remain to be considered the methods of finding the maximum stresses in the main diagonals, verticals, and counters.

43. Lower Half of Main Diagonal.—The stress in the lower half of the end post can be found from the shear in the end panel. The maximum stress in the lower half of any other main diagonal, such as GD , Fig. 27, can be found

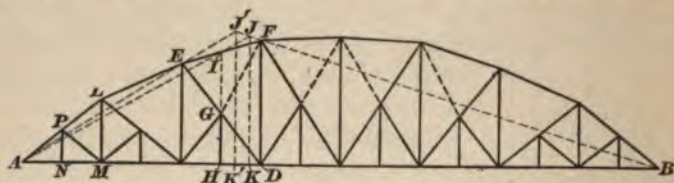


FIG. 27

in precisely the same way as in a diagonal of a simple curved-chord truss. For example, for the member GD , GH may be produced to meet EF at I , the lines AI and BF produced to their intersection at J , and the vertical JK drawn, intersecting the lower chord at K . Then, when there are no loads to the left of H (as is usually the case when the stress in GD is a maximum), the stress in GD is a maximum when $W_r = \frac{KD}{KB} \times \Sigma W$, KB being equal to

$$\frac{BD \times HI}{AH \times DF + BD \times HI}$$

44. Upper Half of Main Diagonal.—The influence line for the upper half of a main diagonal, such as EG , is the same as that for the diagonal ED of a simple curved-chord truss of the same dimensions as that represented in Fig. 27, but having twice the panel length. Then, for EG , the lines AE and BF may be drawn to their intersection J'

and the vertical $J'K'$ drawn to its intersection with the lower chord. The stress in EG is a maximum when $W_{s,p} = \frac{K'D}{K'B} \times \Sigma W$. The influence line for the upper half of the end post PL is the same as that for the shear in the end panel of the simple curved-chord truss shown in Fig. 24; therefore, the stress in PL is a maximum when there is a load at M and the shear in the double panel AM (considering NP and PM to be omitted) is a maximum.

45. Intermediate Verticals.—The stress in an intermediate vertical, such as EC , Fig. 28, is a maximum under the same conditions as the stress in the member EC of the simple curved-chord truss of the same dimensions shown in

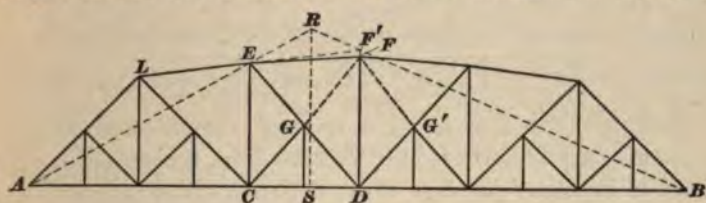


FIG. 28

Fig. 26. If LE is produced to meet FD in F' , the lines AE and BF' drawn to their intersection R , and the vertical RS drawn, the stress in EC is a maximum when

$W_{s,p} = \frac{SD}{SB} \times \Sigma W$, and similarly for other intermediate verticals. The stress in the center vertical is a minimum when the moment at the center of the truss is a maximum; that in the other intermediate verticals may be found most readily by trial in the same way as for the simple curved-chord truss.

46. Counters.—When GF , Fig. 29, is in action as a counter, GD is out of action. Considering $G'F'$ instead, the line $G'H'$ may be produced vertically to its intersection with $E'F'$ at I' , the lines BI' and AF' drawn to their intersection J'' , and the vertical $J''K''$ to its intersection with the lower chord. Then, the stress in $G'F'$ is a maximum when there is a load at H' , and $W_p = \frac{K''H'}{K''B} \times \Sigma W$. When CG

is in action as a counter, EG is in action as a subtie, and the counter CGF is not straight. In finding the position the loads occupy when the stress in CG is a maximum, it may be assumed that CF is straight, and the position of the loads found on this assumption. Then, locating the point K''' in the usual way, the stress in $C'G'$ (considering the right end

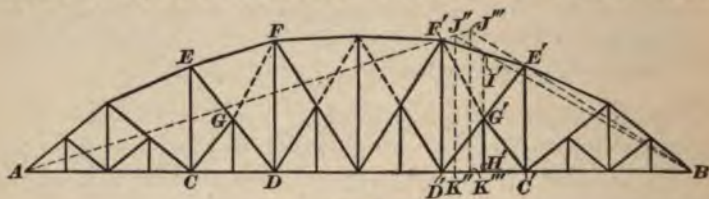


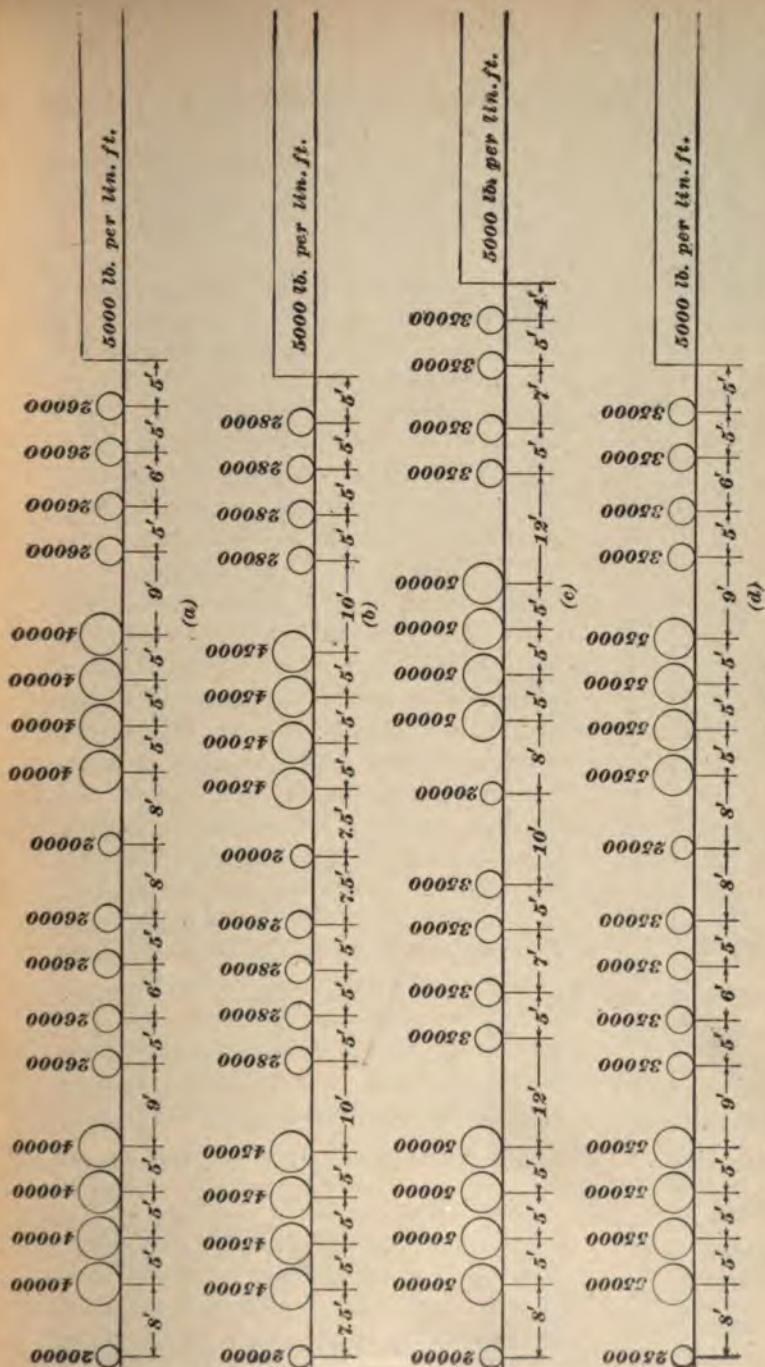
FIG. 29

instead of the left) is a maximum when there is a load at C' and $W_{2p} = \frac{K''' C'}{K''' B} \times \Sigma W$. The ratio $\frac{K''' C'}{K''' B}$ is not strictly correct, but will invariably determine the correct loading. In calculating the stress from this loading, the exact formula given in *Stresses in Bridge Trusses*, Part 3, must be used.

STANDARD SYSTEMS AND METHODS OF COMPUTATION

STANDARD SYSTEMS OF CONCENTRATED LOADS

47. Loads Used in Computations.—The principles used in calculating stresses due to moving concentrated loads find their most frequent applications in connection with railway bridges. Specifications for such bridges require them to be designed to support certain loads, the loading most used consisting of two locomotives coupled together and followed by a train of cars. The actual weight used by different railroads differ; they are usually based on the heaviest rolling stock in use, an allowance being made to provide for a probable increase in weight in the future. As the loads on the wheels of a heavily loaded train are very nearly equal, it is customary to consider the train as a uniform






















<i>A</i>	<i>B</i>	<i>C</i>	<i>D</i>	<i>E</i>	<i>F</i>
		Weights on Axles Pounds	Sums of Distances From First Axle Feet	Sums of Weights on Axles From 1 to <i>N</i> Inclusive Pounds	Moments About Axles of all Preceding Weights Foot-Pounds
1		25,000	0	25,000	0
2		50,000	8	75,000	200,000
3		50,000	13	125,000	575,000
4		50,000	18	175,000	1,200,000
5		50,000	23	225,000	2,075,000
6		32,500	32	257,500	4,100,000
7		32,500	37	290,000	5,387,500
8		32,500	43	322,500	7,127,500
9		32,500	48	355,000	8,740,000
10		25,000	56	380,000	11,580,000
11		50,000	64	430,000	14,620,000
12		50,000	69	480,000	16,770,000
13		50,000	74	530,000	19,170,000
14		50,000	79	580,000	21,820,000
15		32,500	88	612,500	27,040,000
16		32,500	93	645,000	30,102,500
17		32,500	99	677,500	33,972,500
18		32,500	104	710,000	37,360,000
		5,000 <i>x</i>	109	710,000	40,910,000
			109 + <i>x</i>	710,000 + 5,000 <i>x</i>	40,910,000 + 710,000 <i>x</i> + 2,500 <i>x</i> ²

FIG. 32

the multiplier by means of which the first system is derived from the second. The system that is used depends on the amount and class of traffic. For a great many railroads, E40 is heavier than the heaviest traffic; while for some others it has been deemed advisable to use E60 in the design of some of their bridges.

49. Moment Diagram.—When a large number of calculations are to be made from the same system, the work may be shortened by tabulating certain quantities that continually arise. For example, Fig. 32 represents what is called a moment diagram, computed in this case for E50. Opposite *A* are given the numbers of the wheels, the latter being numbered consecutively from the left end; *B* is the spacing of the wheels; *C*, the weight that comes on each axle; *D*, the distance of any wheel from the first; *E*, the sum of all the loads from the first up to and including the one over which the sum is placed; and *F*, the sum of the moments of all the preceding loads about any wheel. The moment about any wheel may be found from the moment about the preceding wheel by adding to the latter the product of the sum of all the loads up to and including that about which the moment is known and the distance between the two wheels. For example, the moment of 1 to 8 about 9 is

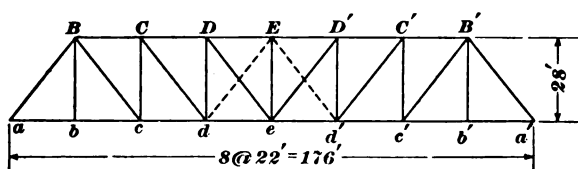


FIG. 33

equal to 8,740,000 foot-pounds. To find the moment about 10, consider that the lever arm of each load from 1 to 9 is increased by 8 feet, and that the moment is increased by the product of 8 feet and the sum of all the loads from 1 to 9, inclusive. Then, the moment about 10 is equal to

$$8,740,000 + 355,000 \times 8 = 11,580,000 \text{ foot-pounds}$$

The use of the diagram will be illustrated by an example.

EXAMPLE.—The truss represented in Fig. 33 supports one-half of a bridge that supports an E50 loading. What is the maximum stress: (a) in the diagonal Bc ? (b) in the chord member CD ?

SOLUTION.—(a) The stress in Bc is a maximum when the shear in the panel bc is a maximum; that is, when $W_p = \frac{P}{l} \times \Sigma W$. In this case, $\frac{P}{l} = \frac{1}{8}$.

When W_2 is at c , a' is 132 ft. to the right of c and 145 ft. to the right of W_1 ; that is, $145 - 109 = 36$ ft. to the right of the head of the uniform load. Then,

$$W = 710,000 + 5,000 \times 36 = 890,000 \text{ lb.}$$

and $\frac{P}{l} \times \Sigma W = 111,250 \text{ lb.}$

The load at b is equal to

$$\frac{\text{moment of 1 and 2 about } c}{2 \times 22} = \frac{575,000}{2 \times 22} = 13,070 \text{ lb.}$$

The left reaction is equal to the moment of loads 1 to 18 and 36 ft. of the uniform load about a' , divided by the length of span. Then, R_1 is equal to

$$\frac{40,910,000 + 710,000 \times 36 + 2,500 \times 36 \times 36}{2 \times 176} = 198,040 \text{ lb. for one truss}$$

The stress in Bc is

$$(198,040 - 13,070) \times \frac{\sqrt{22^2 + 28^2}}{28} = 235,300 \text{ lb., tension. Ans.}$$

(b) The stress in CD is a maximum when the moment at d is a maximum.

When W_9 is at d , $\Sigma W = 710,000 + 5,000 \times 49 = 955,000 \text{ lb.}$, and $\frac{a}{l} \times \Sigma W = 358,125 \text{ lb.}$ As W_l is less than 355,000 lb., this position does not give a maximum.

When W_{10} is at d , $\Sigma W = 710,000 + 5,000 \times 57 = 995,000 \text{ lb.}$, and $\frac{a}{l} \times \Sigma W = 373,125 \text{ lb.}$ As W_l is between 355,000 and 380,000 lb., this position gives a maximum.

When W_{11} is at d , $\Sigma W = 710,000 + 5,000 \times 65 = 1,035,000 \text{ lb.}$, and $\frac{a}{l} \times \Sigma W = 388,125 \text{ lb.}$ As W_l is between 380,000 and 430,000 lb., this position gives a maximum.

When W_{12} is at d , $\Sigma W = 710,000 - 25,000 + 5,000 \times 70 = 1,035,000 \text{ lb.}$, and $\frac{a}{l} \times \Sigma W = 388,125 \text{ lb.}$ As W_l is greater than 405,000 lb., this position does not give a maximum.

It is now necessary to compute the moments when W_{10} and W_{11} , respectively, are at d .

When W_{10} is at d ,

$$R_1 = \frac{40,910,000 + 710,000 \times 57 + 2,500 \times 57 \times 57}{2 \times 176} = 254,240 \text{ lb.}$$

The moment at d is

$$254,240 \times 66 - \frac{11,580,000}{2} = 10,990,000 \text{ ft.-lb.}$$

When W_{11} is at d ,

$$R_1 = \frac{40,910,000 + 710,000 \times 65 + 2,500 \times 65 \times 65}{2 \times 176} = 277,340 \text{ lb.}$$

The moment at d is

$$277,340 \times 66 - \frac{14,620,000}{2} = 10,994,000 \text{ ft.-lb.}$$

Then the stress in CD is

$$10,994,000 \div 28 = 392,600 \text{ lb., compression. Ans.}$$

APPROXIMATE METHODS BY THE USE OF EQUIVALENT LOADS

50. Introduction.—It is very laborious to calculate the stresses in some types of trusses—such as multiple-system and subdivided-panel trusses—if the actual concentrated loads are used. On this account, it is sometimes desirable to substitute for these loads some simpler system of loading that will cause, as nearly as possible, the same stresses. Such loadings are called **equivalent loads**. The two principal kinds of equivalent loads will now be considered.

The difference between the results obtained from equivalent loads and those from the actual wheel loads are relatively greater for short spans than for long spans. For spans up to 75 or 100 feet, the actual wheel loads should be used; and for *all* spans, the stresses in such members as *hip verticals*, *short verticals*, and *diagonals*, and in *floor members* should be calculated from the *actual wheel loads*.

51. Equivalent Uniform Load.—The method of equivalent uniform loads is most useful in computing moments and chord stresses; it consists in substituting for the actual wheel loads a uniform load per linear foot that will cause, as nearly as possible, the same stresses. The best way to compute this equivalent load is to calculate first the maximum moment on the span at a section (usually

Location of Load	ΣW Pounds	$\frac{a}{l} \times \Sigma W$ Pounds	W_l Pounds
W_4 at C	710,000 + 5,000 \times 29 = 855,000	213,750	Less than 175,000
W_5 at C	710,000 + 5,000 \times 34 = 880,000	220,000	Between 175,000 and 225,000
W_6 at C	710,000 + 5,000 \times 43 = 925,000	231,250	Between 225,000 and 237,500
W_7 at C	710,000 + 5,000 \times 48 = 950,000	237,500	Greater than 237,500
W_{11} at C	710,000 - 225,000 + 5,000 \times 75 = 860,000	215,000	Less than 205,000
W_{12} at C	710,000 - 225,000 + 5,000 \times 80 = 885,000	221,250	Between 205,000 and 255,000
W_{13} at C	710,000 - 257,500 + 5,000 \times 85 = 877,500	219,375	Greater than 222,500

called the **quarter point**) mid-way between the center and the end, due to the actual wheel loads, and then find the uniform load over the entire span that will cause the same moment. If w represents the uniform load per linear foot over the entire span, the moment at the quarter point is equal to $\frac{3}{8} w l^2$; if M_c represents the moment at the quarter point due to the actual wheel loads, then

$$\frac{3}{8} w l^2 = M_c$$

and
$$w = \frac{32}{3} \times \frac{M_c}{l^2}$$

It will usually be found that the moment at the center due to this uniform load is slightly greater than that due to the actual wheel loads, and that the moment near the end is slightly less. If the value of w were found from the maximum moment at the center, all the moments would be too small.

EXAMPLE.—What equivalent uniform load may be used in computing chord stresses for a span of 160 feet with an E50 loading?

SOLUTION.—It is first necessary to compute the maximum moment at a section 40 ft. from the left end due to E50. The numerical work can be most conveniently arranged as shown in the accompanying table.

The moment at the quarter point C is a maximum when W_5 , W_6 , and W_{12} , respectively, are at the section.

When W_8 is at C ,

$$R_1 = \frac{40,910,000 + 710,000 \times 34 + 2,500 \times 34 \times 34}{160} = 424,625 \text{ lb.}$$

The moment at C is

$$424,625 \times 40 - 2,075,000 = 14,910,000 \text{ ft.-lb.}$$

When W_8 is at C ,

$$R_1 = \frac{40,910,000 + 710,000 \times 43 + 2,500 \times 43 \times 43}{160} = 475,391 \text{ lb.}$$

The moment at C is $475,391 \times 40 - 4,100,000 = 14,915,640 \text{ ft.-lb.}$

When W_{11} is at C , the wheels 1 to 5 are off the span; and, in using the diagram to get moments about point C and the right end of the span, it is necessary to deduct the moment of these loads as follows: The reaction of R_1 is

$$\frac{(40,910,000 + 710,000 \times 80 + 2,500 \times 80 \times 80) - (2,075,000 + 225,000 \times 166)}{160} = 464,281 \text{ lb.}$$

The moment at C is

$$(464,281 \times 40) - \{16,770,000 - [2,075,000 + (225,000 \times 46)]\} = 14,226,240 \text{ ft.-lb.}$$

The moment is greatest when W_8 is at C ; then,

$$w = \frac{32}{3} \times \frac{14,915,640}{160 \times 160} = 6,214.9, \text{ or, say, } 6,215 \text{ lb. per lin. ft. Ans.}$$

52. Uniform Load and Concentrated Loads.—The method of finding an equivalent system composed of a uniform load and several concentrated loads is most useful in computing shears and web stresses; it consists in substituting for the actual wheel loads a combination of a uniform load, equal to the specified train load, and a single concentrated load for each locomotive, equal to the difference in weight between the locomotive and the weight of a portion of the train the same length as the locomotive, acting simultaneously with the uniform load. For example, in Figs. 31 and 32, wheels 1 to 9 constitute one locomotive, the distance between the end wheels, usually called the **wheel base**, being 48 feet; the second locomotive starts with wheel 10 at a distance of 56 feet from wheel 1, the corresponding wheel of the first locomotive; the latter distance is spoken of as the **length**. The weight of one locomotive, found by adding the loads on all the wheels from 1 to 9, is 355,000 pounds; the weight of an equal length, 56 feet, of train, is $56 \times 5,000 = 280,000$ pounds; the

difference, 75,000 pounds, is taken as a concentrated load acting simultaneously with the uniform load of 5,000 pounds per foot, and is called the **locomotive excess**. When there are two locomotives, there are two excesses; in this case, they are 56 feet apart, the distance between corresponding points of the two locomotives. The equivalent load for one locomotive is shown in

Fig. 34; for convenience in calculation, the concentrated load may be located anywhere with respect to the uniform load. In using this method, it is convenient to find first the

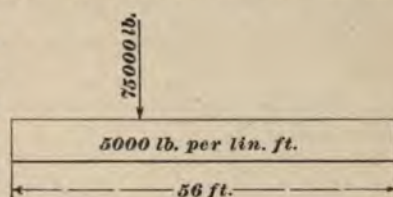


FIG. 34

maximum shears or stresses due to the uniform load, as though it were the only load on the truss, and then those due to the concentrated loads alone, placed so that the shear or stress under consideration will be a maximum; the sum of the results will be the shear or stress desired.

EXAMPLE.—Using the equivalent load found in the preceding paragraph, find the maximum shear in the panel bc of the truss in Fig. 33.

SOLUTION.—Considering first the uniform load alone, the panel load for one truss is $\frac{5,000 \times 22}{2} = 55,000$ lb. The shear in the panel bc due

to a uniform load is a maximum when the joints c to b' are loaded; and, as there are then no panel loads to the left of the panel bc , the shear is equal to the left reaction, or to

$$55,000 \times \frac{(1 + 2 + 3 + 4 + 5 + 6)}{8} = 144,400 \text{ lb.}$$

The shear in the panel bc due to two concentrated loads 56 ft. apart is greatest when one of the loads is at c and the other 56 ft. to the right of c ; and, as there are then no loads to the left of c , the shear in the panel bc is equal to the left reaction, or to

$$\frac{75,000 \times (76 + 132)}{2 \times 176} = 44,300 \text{ lb.}$$

The shear in the panel bc , due to a combination of the uniform load with the locomotive excesses acting simultaneously, is

$$144,400 + 44,300 = 188,700 \text{ lb. Ans.}$$

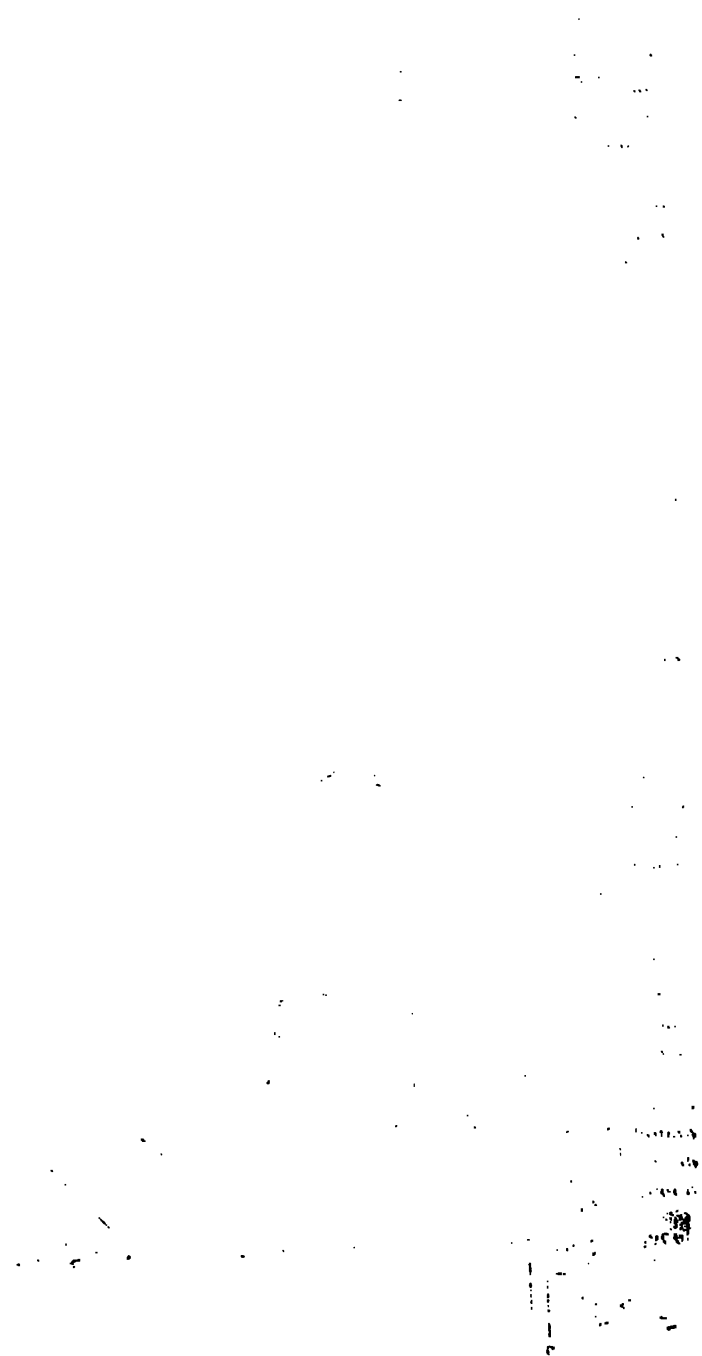
EXAMPLES FOR PRACTICE

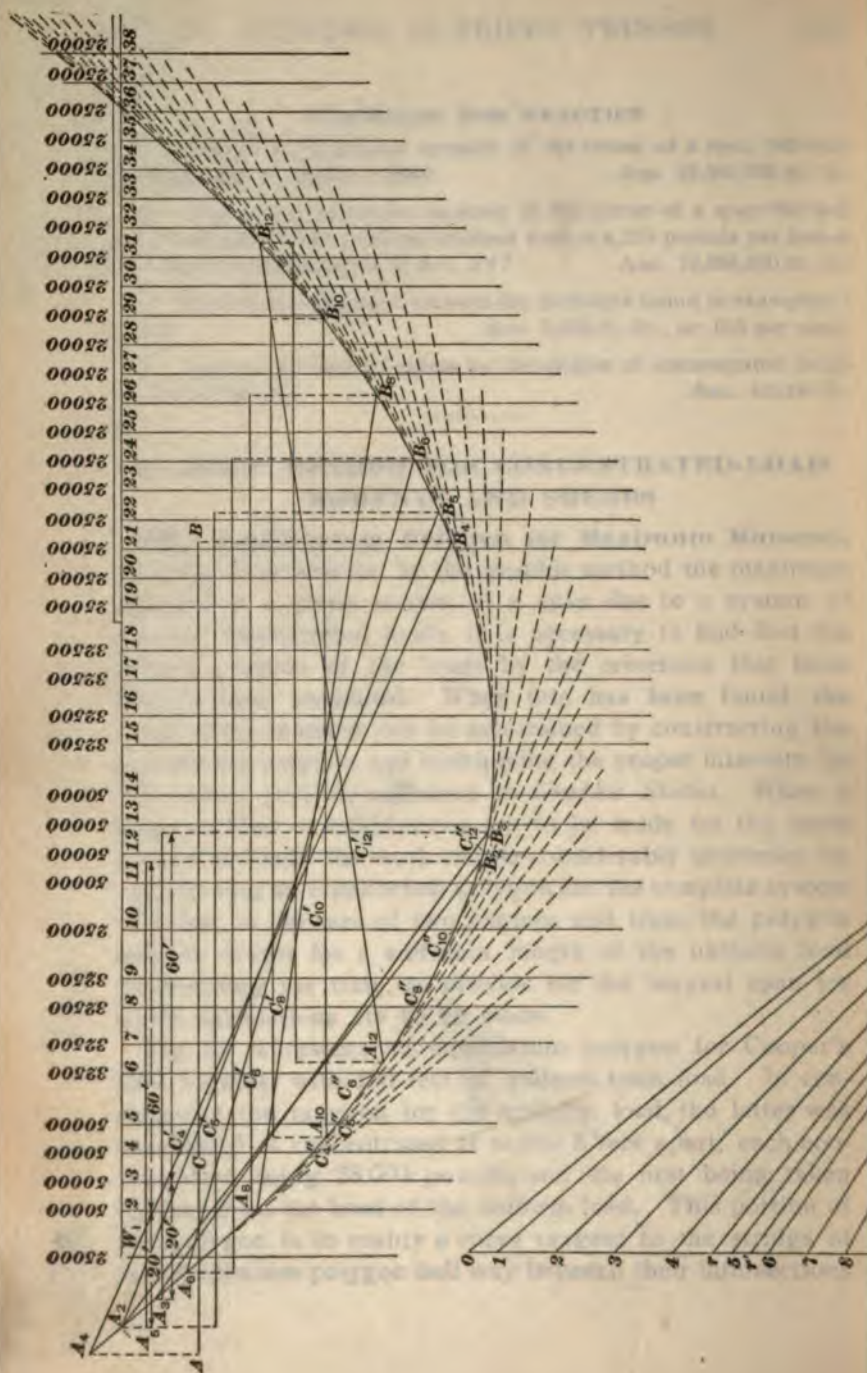
1. What is the maximum moment at the center of a span 160 feet in length due to Cooper's E50? Ans. 19,885,000 ft.-lb.
2. What is the maximum moment at the center of a span 160 feet in length, due to the equivalent uniform load of 6,215 pounds per linear foot found in the example in Art. 51? Ans. 19,888,000 ft.-lb.
3. What is the difference between the moments found in examples 1 and 2? Ans. 3,000 ft.-lb., or .015 per cent.
4. Find the locomotive excess for the system of concentrated loads shown in Fig. 21. Ans. 42,000 lb.

GRAPHIC METHOD FOR CONCENTRATED-LOAD
MOMENTS AND SHEARS**53. Equilibrium Polygon for Maximum Moment.**

In order to determine by the graphic method the maximum moment at a given section of a span due to a system of moving concentrated loads, it is necessary to find first the correct position of the loads by the principles that have already been explained. When this has been found, the value of the moment can be ascertained by constructing the equilibrium polygon and multiplying the proper intercept by the normal ray, as explained in *Graphic Statics*. When a large number of calculations are to be made for the same system of loads, the work can be considerably shortened by constructing an equilibrium polygon for the complete system of loads; in the case of two engines and train, the polygon may be drawn for a sufficient length of the uniform load representing the train, to provide for the longest span for which calculations are to be made.

Fig. 35 represents an equilibrium polygon for Cooper's E50, together with 100 feet of uniform train load. In constructing the polygon for the uniform load, the latter was considered as concentrated at points 5 feet apart; each concentration being 25,000 pounds, and the first being taken 2.5 feet from the head of the uniform load. This portion of the polygon is in reality a curve tangent to the strings of the equilibrium polygon half way between their intersections





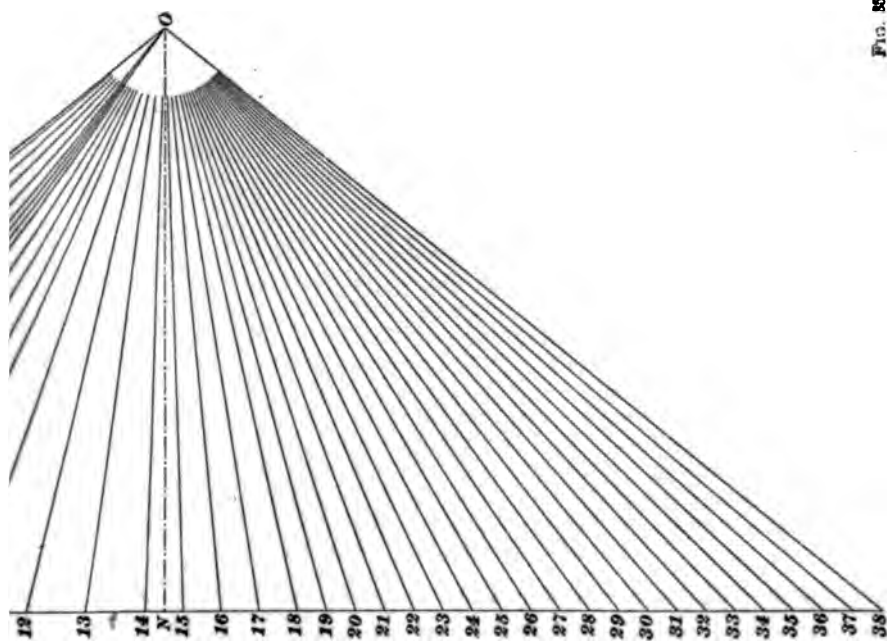


Fig. 55

with the assumed concentrations. In practice, the polygon is made as large as possible; if great care is taken in scaling all distances, the results will be sufficiently close.

54. Method of Procedure.—If it is known what position the loads occupy when the moment is a maximum at a given section C , the value of the moment can be found by laying off the span to the same scale as the spacing of the loads, placing section C at the proper load and drawing verticals through the ends of the span. These verticals are the lines of action of the reactions; then, if the closing line is drawn between their intersections with the strings of the equilibrium polygon, and a vertical line is drawn through C , the moment at C is equal to the intercept of this vertical multiplied by the normal ray.

If it is not known what position the loads occupy when the moment is a maximum, the span may be laid off with the section C at several of the loads successively, and the preceding process repeated until the maximum intercept is found.

EXAMPLE.—Let it be required to find the maximum moment at the quarter point of a span 140 feet long due to Cooper's E50.

SOLUTION.—As it is known that the span will be quite fully loaded, it may be assumed that there will be several loads to the left of the quarter point. The span may be laid off, to the same scale as the spacing of the loads, on the edge of a straight strip of paper, and the quarter point marked; this strip may then be laid horizontally on the diagram, Fig. 35, so that C is at one of the loads, for instance W_4 ; verticals through A and B cut the equilibrium polygon at A_4 and B_4 , respectively; then A_4B_4 is the closing line for this position. A vertical through C (the line of action of W_4) gives the intercept $C_4'C_4''$, and the moment at C is equal to $C_4'C_4'' \times N$. Next, C may be placed at W_5 , then at W_6 , etc., until the maximum intercept is found. In this case, the intercept increases until W_5 is at C , then decreases and increases again until W_{12} is at C . The intercept is a maximum when W_5 is at C , and is equal, by scale, to 23.52 ft. The normal ray is equal to 500,000 lb., by construction. Then, the maximum moment at C is

$$23.52 \times 500,000 = 11,760,000 \text{ ft.-lb.} \quad \text{Ans.}$$

55. Maximum Shear at a Given Section.—The maximum shear at a given section of a beam, due to a

system of moving concentrated loads, can be found in almost the same way as the moment, by trying several loads at the section successively, and drawing the closing line of the equilibrium polygon and the ray parallel to that line in the force polygon for each position of the loads. Then, the reactions and the shear at any section can be scaled directly from the load line.

EXAMPLE.—Let it be required to find the maximum shear at the quarter point of a beam 80 feet in length, due to Cooper's E50.

SOLUTION.—The span may be laid off with the quarter point at W_1 , and verticals drawn at the ends of the span, intersecting the equilibrium polygon at A_1 and B_1 ; the closing line A_1B_1 is then drawn, Fig. 35. Then, if Or is drawn parallel to A_1B_1 , the left reaction is equal to $0-r$, and the shear at the quarter point $1-r = 179,000$ lb. Next, the span may be laid off with the quarter point at W_2 ; the left reaction is equal to $0-r'$, and the shear at the quarter point is $0-r' - 0-2 = 2-r' = 159,000$ lb. As the shear when W_1 is at the quarter point is greater than this, the maximum shear is 179,000 lb. Ans.

STRESSES IN BRIDGE TRUSSES

(PART 5)

TRANSVERSE FORCES

1. Introduction.—All the outer forces considered in *Stresses in Bridge Trusses*, Parts 2, 3, and 4, are vertical forces. As explained in Part 1, bridges are subject to the action of horizontal or transverse forces—principally wind pressure and centrifugal force—that must be resisted by lateral trusses or bracing lying in planes that are not vertical. The magnitudes of these transverse forces and the stresses caused by them will now be discussed.

WIND PRESSURE

2. Intensity of Wind Pressure.—Records of the intensity of wind pressure have been kept in various places, and from them it has been deduced that the maximum pressure during heavy gales may be as high as 50 pounds per square foot of exposed surface; in some instances, even greater pressures have been recorded, but they are probably of so rare occurrence that they may be safely neglected. The best bridge engineers of the present time consider it sufficient to provide for a pressure of 50 pounds per square foot over the entire exposed area of railroad and highway bridges, or, in the case of railroad bridges, an alternative pressure, if it produces greater stresses in the lateral system, of 30 pounds per square foot against both the exposed surface of a train of cars and the exposed area of the

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bridge. The weight of a train of empty cars is approximately 900 pounds per linear foot, and the standard distance from center to center of rails is about 4.9 feet. Taking moments about the top of a rail, as shown in Fig. 1, the moment of stability of an empty car, or the resistance to overturning, is

$$900 \times \frac{4.9}{2} = 900 \times 2.45 = 2,205 \text{ foot-pounds per linear foot}$$

The exposed area of the side of a car will be assumed to extend from about 2 feet to 12 feet above the top of the rail; then, the exposed area per linear foot is $10 \times 1 = 10$ square feet, the center of which is 7 feet above the top of the rail,

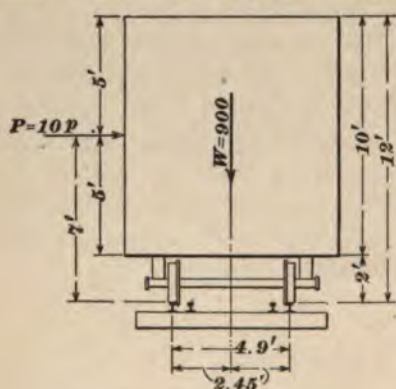


FIG. 1

as shown in Fig. 1. Then, if the wind pressure per square foot, uniformly distributed over the side of the car, that will just overturn it is represented by p , the total pressure P per linear foot is $10p$, and, taking moments about the top of the rail, the overturning moment is $10p \times 7$ foot-pounds per linear foot. Placing this equal to the moment of stability just

found, and solving for p , we have

$$p = \frac{2,205}{10 \times 7} = 31.5 \text{ pounds per square foot}$$

It is assumed, therefore, that when the wind pressure is greater than about 30 pounds per square foot it will be impossible to operate trains; the higher pressure of 50 pounds per square foot is assumed to act against the unloaded bridge if it causes greater stresses in the members of the lateral system than does a pressure of 30 pounds per square foot. In case a train of cars is caught on a bridge by a sudden gust of wind, the pressure may be greater than 30 pounds per square foot; but, as this condition will probably last for

a very short time, it is assumed that no harm will result from it. For all practical purposes, the pressure on a train of cars may be taken equal to 300 pounds per linear foot, acting 7 feet above the top of the rail, although other values and distances are used by some engineers.

In referring to the direction of the wind, it will be convenient to speak of the *windward truss* and the *leeward truss*, the former being the one on the side from which the wind is blowing, and the latter being the one on the other side. For example, if the bridge is located north and south, and the wind is blowing from east to west, the east truss is the windward truss, and the west truss is the leeward truss.

3. Exposed Area.—The exposed area of an unloaded bridge is usually taken equal to twice the exposed area of one truss; it is found by multiplying the length of each truss member by its greatest width, as seen in elevation, multiplying the sum of the products by 2, and adding the product to the area of the floor, as seen in elevation. The exposed area of a loaded through bridge is usually taken equal to the sum of the exposed area of one truss and that of the elevation of the floor, as it is assumed that the leeward truss is sheltered by the train of cars. In deck bridges, the exposed area of truss and floor is taken the same as for an unloaded bridge, since the train of cars does not shelter the leeward truss. In highway bridges, the pressure on the exposed area of the loads is usually small compared with that on the exposed area of the bridge, and hence in practice is quite customarily neglected, or else a small amount is added to the pressure on the loaded chord. In railroad bridges, the pressure against the exposed area of a train of cars is relatively large, and is usually taken into account. The pressure against the trusses is taken as a fixed load covering the whole length of span; that against the train is taken as a moving load covering such portion of the span as will cause maximum stresses in the members of the lateral system.

There are other methods of computing the wind pressure on bridges, such as allowing a fixed amount per linear foot

of each chord, but the method just outlined is most generally used and is believed to be superior to any other. For this reason, it will be used in this and succeeding Sections.

4. Lateral System.—The wind pressure against the exposed surface of a bridge is transmitted to the joints of the trusses by the members themselves, and is resisted by bracing, called **lateral bracing**, connecting the main trusses and lying in the planes of the upper and lower chords, respectively, and by vertical or inclined bracing, called **sway bracing** or **transverse bracing**, connecting opposite posts in the two trusses. The sway bracing connecting the end posts is called the **portal bracing**. The combined system formed by all these bracings is usually called the **lateral system**. The pressure against the exposed surfaces of the loads is transmitted to the lateral system of the loaded chord by means of the floor.

LATERAL BRACING

5. Description.—The lateral bracing usually consists of Pratt trusses lying in the planes of the chords of the main trusses. In parallel-chord bridges, the lateral trusses lie in horizontal planes; in inclined-chord bridges, the lateral trusses of the inclined chord lie in the several planes of the inclined members, and the panel lengths of the lateral trusses are equal to the actual lengths of the respective chord members. For the purpose of determining the stresses, however, it is customary and sufficiently accurate for all ordinary spans to consider the panels of the lateral truss equal in length to those of the main trusses; the stresses are then found in the same way as for parallel-chord trusses. In Fig. 2, (*b*) is the main truss; (*a*), the upper lateral truss; and (*c*), the lower lateral truss, *b* being the distance center to center of trusses. The chords *BC*, *CD*, etc. of the lateral trusses are the respective chord members of the main trusses; the transverse members *CC*., *DD*., etc. are compression members and connect the opposite joints of the two main trusses; the diagonals *CB*., *DC*.,

etc. are considered to be tension members, although in the best modern practice they are built to resist both tension and compression. In a through bridge, the floorbeams act as the transverse struts for the lower lateral system; in a deck bridge having a floor system, the floorbeams act as transverse struts for the upper lateral system.

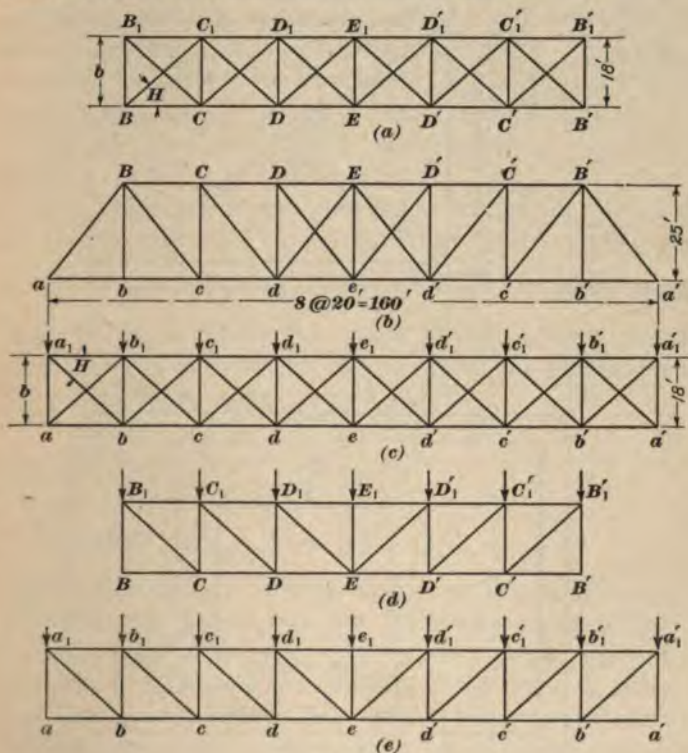


FIG. 2

There are two diagonals in each panel of the lateral trusses in order to resist the wind pressure from either direction; when one set of diagonals is in action, the stresses in the others are assumed to be zero. Assuming that the wind panel loads act as shown in Fig. 2 (c), the diagonals shown in (d) and (e) will be in action. The loads on the lower lateral truss are transmitted directly to the abutments or other

supports; the loads on the upper lateral truss are transmitted to the upper joints of the end posts, which, with the assistance of the portal bracing, transmit them to the supports.

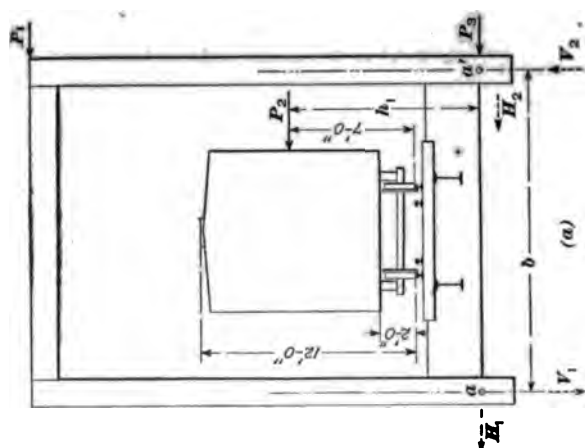
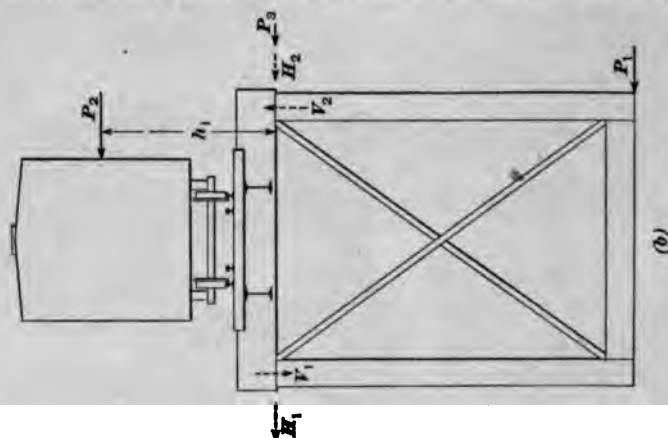


FIG. 3

6. Calculation of Wind Stresses.—In Fig. 3, (a) is the cross-section of a through bridge, and (b) the cross-section of a deck bridge; P_1 , P_2 , and P_3 are the wind panel loads on the unloaded chord, train of cars, and loaded chord, respectively. It will be assumed that the lateral trusses of the unloaded chords for the two types of bridges here

represented are in the planes of the top and bottom, respectively, of the chord members. It will also be assumed that the lateral trusses of the loaded chords lie in the planes of the bottoms of the floorbeams. The pressure P_s will evidently cause stresses in the lateral trusses of the loaded chord; it also tends to overturn the train, thereby increasing the amount of the live load that goes to the leeward truss and decreasing the load on the windward truss. The pressure may be resolved into the components H_1 and V_1 at a , Fig. 3 (a), and H_2 and V_2 at a' , a and a' being the intersections of the lateral truss of the loaded chord with the main vertical trusses. Then,

$$P_s = H_1 + H_2$$

It is impossible to compute the actual values of H_1 and H_2 by the equations of equilibrium, but these values may be assumed equal to each other and to $\frac{P_s}{2}$. In actual practice, however, P_s is frequently added to P_s , and the total wind panel load is assumed to be applied at the windward side. Taking moments about a' , we have

$$V_1 b = P_s h_1;$$

whence

$$V_1 = \frac{P_s h_1}{b}$$

Similarly,

$$V_2 = \frac{P_s h_1}{b} = V_1$$

The component V_2 simply decreases the panel loads and, therefore, the stresses in the windward truss; if only the maximum stresses are sought, it need not be further considered; V_1 increases the stresses in the leeward truss, and this increase may be found by multiplying the maximum live-load stresses due to vertical loads by the ratio of V_1 per panel to the live panel load; for, under any condition of loading, the live panel load on the leeward truss may be increased by the overturning effect of the wind. Then, if W''' is the live panel load, the wind stresses in the mem-

bers of the vertical truss are equal to $\frac{P_s h_1}{W'''} b$ multiplied by the maximum live-load stresses. The stresses in the lateral

trusses may be found in the same way as for a vertical Pratt truss, as explained in *Stresses in Bridge Trusses*, Part 2. The overturning effect due to the wind pressure on the upper chord will be discussed later in connection with the portal bracing.

EXAMPLE.—Assume that the eight-panel through bridge shown in Fig. 2 (*b*) is subject to a wind pressure of 400 pounds per linear foot on the upper chord; and a fixed, or dead, wind pressure of 200 pounds per linear foot, and a live, or moving, wind pressure of 300 pounds per linear foot, on the lower chord. What are the maximum wind stresses in all the members: (*a*) of the upper lateral truss? (*b*) of the lower lateral truss? (*c*) What is the amount by which each panel load of the leeward truss must be increased, assuming that the distance from the center of the side of a car to the lower lateral truss is 11 feet, and the distance from center to center of the trusses is 18 feet?

SOLUTION.—Since the panel length of the bridge is 20 ft., Fig. 2 (*b*), and the distance center to center of trusses is 18 ft., the length of a diagonal of the lateral truss is

$$\sqrt{20^2 + 18^2} = 26.91 \text{ ft., and } \csc H = \frac{26.91}{18}$$

(*a*) The wind panel load for the upper lateral truss is $400 \times 20 = 8,000$ lb.; the reactions are each

$$\frac{8,000 \times 5}{2} = 20,000 \text{ lb.}$$

Then, the stresses in the members are as follows [Fig. 2 (*d*)]:

MEMBER	STRESS, IN POUNDS
$B_1 C$	$20,000 \times \frac{26.91}{18} = -29,900$
$C_1 C$	$+20,000$
$C_1 D$	$(20,000 - 8,000) \times \frac{26.91}{18} = -17,900$
$D_1 D$	$(20,000 - 8,000) = +12,000$
$D_1 E$	$(20,000 - 8,000 - 8,000) \times \frac{26.91}{18} = -5,980$
$E E_1$	$+8,000$
$B C$	$= 0$
$B_1 C_1$	$\frac{20,000 \times 20}{18} = +22,200$
$C D$	$\frac{20,000 \times 20}{18} = -22,200$
$C_1 D_1$	$\frac{20,000 \times 40 - 8,000 \times 20}{18} = +35,600$
$D E$	$\frac{20,000 \times 40 - 8,000 \times 20}{18} = -35,600$
$D_1 E_1$	$\frac{20,000 \times 60 - 8,000 \times (20 + 40)}{18} = +40,000$

(b) The dead wind panel load for the lower lateral truss is $200 \times 20 = 4,000$ lb.; the reactions are each

$$\frac{4,000 \times 7}{2} = 14,000 \text{ lb.}$$

The live wind panel load is $300 \times 20 = 6,000$ lb.; each live reaction for full load on the lower lateral truss is

$$\frac{7 \times 6,000}{2} = 21,000 \text{ lb.}$$

The dead- and live-load stresses are found as explained in *Stresses in Bridge Trusses*, Part 2; in the present case they are found together. The combined stresses, in pounds, are as follows [Fig. 2 (e)]:

The combined stress in a, b is

$$(21,000 + 14,000) \times \frac{26.91}{18} = -52,300$$

The combined stress in b, b is

$$21,000 + 14,000 = +35,000$$

The combined stress in b, c is

$$\left[(14,000 - 4,000) + \frac{6,000 \times \frac{6 \times (6+1)}{2}}{8} \right] \times \frac{26.91}{18} = -38,500$$

The combined stress in c, c is

$$\left[(14,000 - 4,000) + \frac{6,000 \times \frac{6 \times (6+1)}{2}}{8} \right] = +25,800$$

The combined stress in c, d is

$$\left[(14,000 - 4,000 - 4,000) + \frac{6,000 \times \frac{5 \times (5+1)}{2}}{8} \right] \times \frac{26.91}{18} = -25,800$$

The combined stress in d, d is

$$\left[(14,000 - 4,000 - 4,000) + \frac{6,000 \times \frac{5 \times (5+1)}{2}}{8} \right] = +17,250$$

The combined stress in d, e is

$$\left[(14,000 - 4,000 - 4,000 - 4,000) + \frac{6,000 \times \frac{4 \times (4+1)}{2}}{8} \right] \times \frac{26.91}{18} = -14,200$$

The combined stress in e, e is

$$\text{full panel load} = 4,000 + 6,000 = +10,000$$

The combined stress in a, b is 0.

The combined stress in a, b, c is

$$\frac{35,000 \times 20}{18} = +38,900$$

The combined stress in b, c is

$$\frac{35,000 \times 20}{18} = -38,900$$

The combined stress in $b_1 c_1$ is

$$\frac{35,000 \times 40 - 10,000 \times 20}{18} = +66,700$$

The combined stress in $c d$ is

$$\frac{35,000 \times 40 - 10,000 \times 20}{18} = -66,700$$

The combined stress in $c_1 d_1$ is

$$\frac{35,000 \times 60 - 10,000 \times (20 + 40)}{18} = +83,300$$

The combined stress in $d e$ is

$$\frac{35,000 \times 60 - 10,000 \times (20 + 40)}{18} = -83,300$$

The combined stress in $d_1 e_1$ is

$$\frac{35,000 \times 80 - 10,000 \times (20 + 40 + 60)}{18} = +88,900$$

(c) The live panel load (which is the wind pressure on the train) was found in (b) to be 6,000 lb. Since the trusses are 18 ft. apart, and the distance from the center of the wind pressure on the train to the lower lateral truss is 11 ft., the increase in the vertical load per panel on the leeward truss is

$$\frac{6,000 \times 11}{18} = 3,670 \text{ lb.}$$

EXAMPLES FOR PRACTICE

1. A ten-panel deck bridge with vertical end posts has a span length of 180 feet and a distance from center to center of trusses of 18 feet. If the wind pressure on a train of cars is 300 pounds per linear foot, and on the upper chord is 300 pounds per linear foot, what is the maximum stress in the diagonal of the second panel of the upper lateral truss?

Ans. - 54,200 lb.

2. If the center of the wind pressure on the cars on the bridge referred to in example 1 is 10 feet above the upper lateral truss, what is the amount by which each panel load of the leeward truss must be increased to allow for the overturning effect of the wind?

Ans. 3,000 lb.

3. If the wind pressure on the lower chord of the bridge referred to in example 1 is 200 pounds per linear foot, what is the maximum stress in the second panel of the leeward chord of the lower lateral truss?

Ans. - 16,200 lb.

PORTAL AND OTHER SWAY BRACING

7. Description.—It was explained in Art. 5 that the lateral truss of the upper chord simply transmits the wind pressure to the upper joints of the end posts, usually called

the **hip joints**, and that the end posts, with the assistance of the portal bracing, transmit it to the supports. In the through bridge, the portal bracing lies in the plane of the end posts, usually inclined, and is made as deep as possible without encroaching on the headroom required by the loads that cross the bridge. In Fig. 4 are represented the end posts AC and $A'C'$ of two trusses connected by the standard types of portal bracing, usually called simply

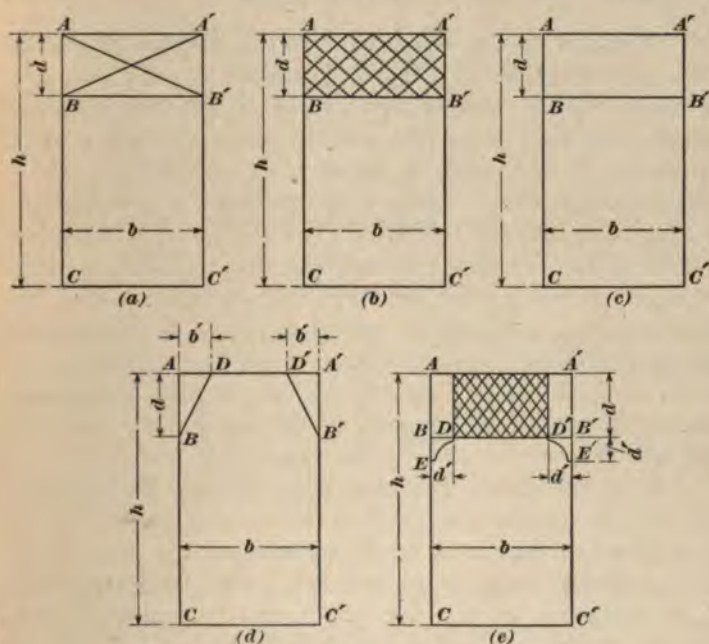


FIG. 4

the *portal*: (a) consists of the transverse members AA' and BB' and the two diagonals AB' and $A'B$; (b) consists of the transverse members AA' and BB' and a lattice web; (c) consists of a plate girder; and (d) consists of the transverse member AA' and the **knee braces** BD and $B'D'$. In (e), the portion $AA'B'B$ may be similar to either of the forms shown in (b) and (c); the brackets BDE and $B'D'E'$ under the portal are frequently added, partly for architectural

effect, and partly to afford additional stiffness. It is almost impossible to state in a general way which type should be used, as this will depend to a great extent on the judgment of the designer; in the analysis of each type, however, the conditions to which that type is best adapted will be mentioned. In all that follows regarding the stresses in the portal bracing, it must be remembered that the portal and all the stresses considered lie in the plane that passes through the center lines of the end posts.

8. Method of Calculation.—In the analysis of trusses in the preceding articles, it was assumed in each case that the stress in each member was a direct stress, and that each member was hinged or free to turn about the joints at its two ends. In the case of portal bracing, the end post, in addition to the direct stresses, is subjected to shearing and bending stresses due to the wind pressure; and, whenever the end posts are considered cut by a plane of section, these additional stresses must be taken into consideration. The portal and end posts can be considered as a structure some members of which are subjected to bending as well as to direct stresses. The only external forces that act on this structure are the wind pressure at the top and the reactions that this pressure causes at the supports C and C' . As in the case of the lateral truss, the wind pressure on the portal may, for convenience, be assumed to act as a single force on the windward side; it is equal to one-half the sum of the wind panel loads on the upper chord, including the pressure at the hip joint; in this Section, it will be called P . The analytic method is the most convenient for finding the stresses in the portal and end posts, and so the graphic method will not be discussed.

9. Reactions.—The reactions at C and C' , Fig. 5 (*a*), may be found by applying the conditions of equilibrium to the external forces, considering the reactions to be resolved into components parallel and at right angles, respectively, to the end posts; the components parallel to the end posts will be called the Y components, and those perpendicular to the

end posts the X components. Assuming the components X and X' , Y and Y' of the reactions at C and C' to act as shown in Fig. 5, and denoting by b the distance from center

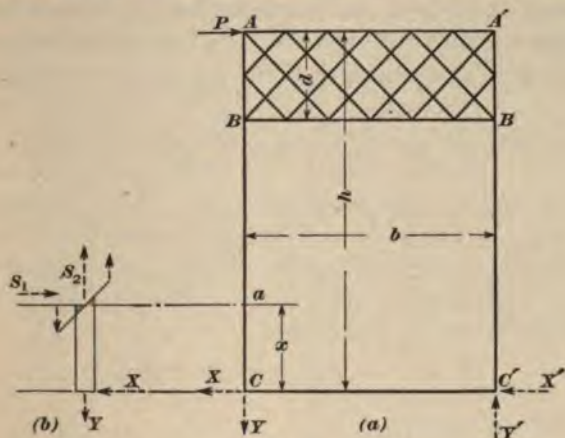


FIG. 5

to center of the trusses, and by h the distance from the top of the end post to the bottom, we have, from the equation $\Sigma M = 0$,

$$Y = \frac{P \times h}{b}, \quad Y' = \frac{P \times h}{b} = Y;$$

and from the equation $\Sigma X = 0$,

$$P = X + X'$$

It is impossible to calculate the actual values of X and X' by the equations of equilibrium, but in practice it is customary in this case to assume that they are each equal to $\frac{P}{2}$. This is probably quite close to their actual values. The reactions as just found are the same, no matter what type of portal is used.

10. End Posts.—The stresses in the end posts and the portals depend on the condition of the connections at C and C' —whether the end posts are hinged at these points or are firmly fixed in direction. For the present, it will be assumed that they are hinged, and the stresses will be found on this assumption. Later on, the effect of fixing them will be

discussed. As there are no diagonals meeting the end posts at C and C' , Fig. 5 (a), those posts must be capable of resisting both the X and Y components: the Y components will act along them, causing direct stresses; the X components will act on the portions BC and $B'C'$, below the portal, as on overhanging beams loaded at C and C' , and fixed in direction at B and B' , respectively.

The end post BC may be considered cut by a plane at the section a at a distance equal to x from C , and the portion between a and C considered as a free body, as shown in Fig. 5 (b). As this portion is in equilibrium, the forces acting on it must be in equilibrium. Let S_1 and S_2 be, respectively, the horizontal and the vertical component of the stress at a . Then, from the equation $\Sigma Y = 0$,

$$S_2 = Y = P \frac{h}{b}, \text{ tension in } BC$$

From the equation $\Sigma X = 0$,

$$S_1 = X = \frac{P}{2}, \text{ shear on section } a$$

Applying the equation $\Sigma M = 0$, moments being taken about the center of section a , the moment of Y will be zero, and the moment of resistance to be offered by the end post at section a will equal Xx , or $\frac{Px}{2}$. Therefore, there is tension in BC from B to C equal to $P \frac{h}{b}$, shear at any section between B and C equal to $\frac{P}{2}$, and bending moment at any distance x from C equal to $\frac{Px}{2}$; at B , where x is equal to $h - d$, this moment is greatest and equal to $\frac{P}{2} (h - d)$. In like manner, it may be shown that in member $B'C'$ there is compression equal to $P \frac{h}{b}$, shear at any point between B' and C' equal to $\frac{P}{2}$, and bending moment at B' equal to $\frac{P}{2} (h - d)$. The fiber stresses caused in the end posts by

these stresses must be added to the stresses due to dead and live loads to get the maximum stresses.

The stresses from C and C' to the lowest point of the portal bracing will be the same as just given, no matter what type of portal is used. The portals will now be considered.

11. The Braced Portal.—The portal shown in Fig. 4 (*a*) is sometimes called a **braced portal**, and is used when the depth of portal is large compared to the length, thereby

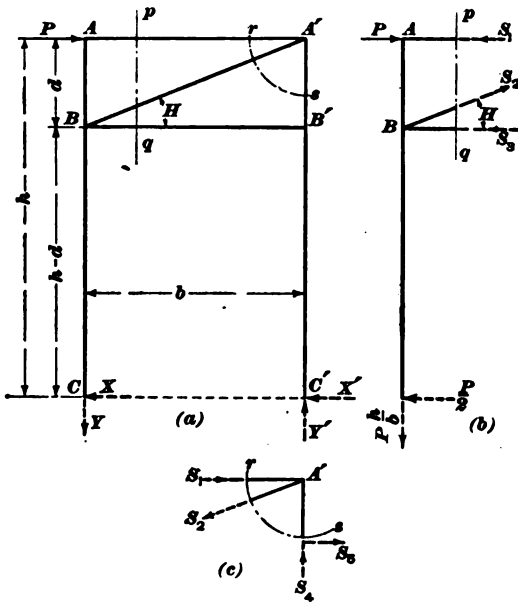


FIG. 6

giving the diagonals a good inclination. The latter are usually constructed in such a way that they can resist tension and compression, but it is customary to assume that they act in tension only; when one is in action, the other is assumed to be out of action. If the wind comes from the left, P is applied at A , and the diagonal BA' is in action, as shown in Fig. 6 (*a*). The stresses in the members may now be found by the method of moments and shears. The portal may be considered cut by a plane, such as pq , Fig. 6 (*a*), that cuts

AA' , BA' , and BB' , and the portion to the left of the plane considered as a free body, as shown in Fig. 6 (*b*); the forces acting on this portion are P at A , $\frac{P}{2}$ and $\frac{Ph}{b}$ at C , and the forces S_1 , S_2 , and S_3 representing the stresses in AA' , BA' , and BB' , respectively, and assumed to act as represented in Fig. 6 (*b*). Applying the equation $\sum Y = 0$, we have

$$S_2 \sin H - \frac{Ph}{b} = 0;$$

whence

$$S_2 = \frac{Ph}{b} \csc H$$

As this is positive, the assumed direction of S_2 is correct, and the stress in BA' is tension.

The stress in AA' , represented by S_1 , can be found most readily by the method of moments, taking B as the center of moments. Applying the equation $\sum M = 0$, we have

$$\frac{P}{2}(h-d) + Pd - S_1 d = 0;$$

whence

$$S_1 = \frac{Ph}{2d} + \frac{P}{2}$$

As this is positive, the assumed direction of S_1 is correct, and the stress in AA' is compression.

The stress in BB' , represented by S_3 , can be found most readily by applying the equation $\sum X = 0$ to all the X components that act on the portion shown in Fig. 6 (*b*). The X component of S_2 is $S_2 \cos H$, or $\frac{Ph}{b} \times \cot H$, and, substituting for $\cot H$ its value $\frac{b}{d}$, $S_2 \cos H$ becomes $\frac{Ph}{d}$. Then,

$$S_3 - \frac{Ph}{d} - P + \left(\frac{Ph}{2d} + \frac{P}{2}\right) + \frac{P}{2} = 0;$$

whence

$$S_3 = \frac{Ph}{2d}$$

As this is positive, the assumed direction of S_3 is correct, and the stress in BB' is compression.

At the joint A , the two external forces P and S_1 are not equal; the difference between them,

$$\left(\frac{Ph}{2d} + \frac{P}{2}\right) - P = \frac{Ph}{2d} - \frac{P}{2},$$

acting to the left, causes shearing stress in the member AB . As there are no Y forces at the joint A , there is no direct stress in AB .

The joint A' may be considered a free body, as represented in Fig. 6 (c); the forces acting on it are S_1 , S_2 , S_3 , and S_4 , the latter representing the shear in $A'B'$. Applying the equation $\sum Y = 0$, we have

$$S_4 - S_2 \sin H = 0, \text{ or } S_4 - \frac{Ph}{b} = 0;$$

whence

$$S_4 = \frac{Ph}{b}$$

As this is positive, the assumed direction of S_4 is correct, and the direct stress in $A'B'$ is compression. Applying the equation $\sum X = 0$, we have

$$S_4 + S_1 - S_2 \cos H = 0$$

or

$$S_4 + \left(\frac{Ph}{2d} + \frac{P}{2} \right) - \frac{Ph}{d} = 0;$$

whence

$$S_4 = \frac{Ph}{2d} - \frac{P}{2}, \text{ shear in } A'B'$$

which is equal to the shear in AB .

When the wind blows from the right, the diagonal AB' will be in action, and $A'B$ out of action; the stresses in the other members of the portal will be the same as those given above; the stresses in the end posts will change in character.

12. Plate-Girder Portal.—Fig. 4 (c) shows a type of portal composed of the two transverse members AA' and BB' , usually made of angles, connected by a solid web or plate. The construction is identical with that of a plate girder. This form of portal is called a **plate-girder portal**, and is especially adapted to cases where the available depth of portal is small compared with the length, as in the case of a shallow truss. In this Section, it will be sufficient to get the expressions for the shears and moments on the girder at various points; the methods of calculating the flange and web stresses from them will be fully discussed in connection with the design of plate girders. If the plate-girder portal shown in Fig. 7 is cut by a plane at

right angles to the flanges AA' and BB' at a distance x from AC , and the portion to the left of the section is considered as a free body, it will

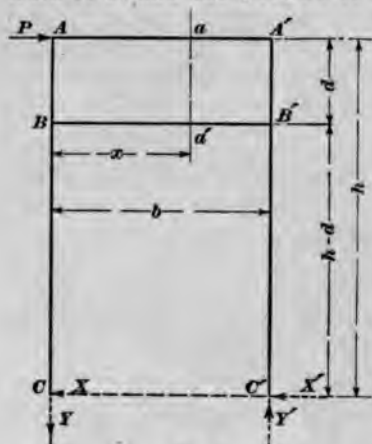


FIG. 7

be seen that the shear on the section is equal to Y , which is equal to $P\frac{h}{b}$; and, as this is independent of x , the shear will be constant from AB to $A'B'$. When the wind comes from the left, the shear is negative; when from the right, it is positive.

The moment M of the external forces on the left of the section about the point a , which may be called the bend-

ing moment at a in the upper flange, is equal to

$$Xh - Yx = \frac{Ph}{2} - \frac{Phx}{b}$$

$$\text{When } x = 0, \quad M = \frac{Ph}{2}$$

$$\text{When } x = \frac{b}{2}, \quad M = \frac{Ph}{2} - \frac{Ph}{2} = 0$$

$$\text{When } x = b, \quad M = \frac{Ph}{2} - Ph = -\frac{Ph}{2}$$

The moment M' about the point a' , or the bending moment at a' in the lower flange, is equal to

$$X(h-d) - Yx + Pd = \frac{Ph}{2} + \frac{Pd}{2} - \frac{Phx}{b}$$

$$\text{When } x = 0, \quad M' = \frac{Ph}{2} + \frac{Pd}{2}$$

$$\text{When } x = \frac{b}{2}, \quad M' = \frac{Pd}{2}$$

$$\text{When } x = b, \quad M' = -\frac{Ph}{2} + \frac{Pd}{2}$$

When the wind comes from the right, the distance x is the distance from the section to the right end, and the moments are given by the same formulas.

13. Lattice Portal.—Fig. 4 (*b*) shows a type of portal composed of the two transverse struts AA' and BB' connected by an open or lattice web, there being several systems of web members. This is by far the most common type in use, especially where a reasonable depth may be had—say, at least, one-sixth the distance from center to center of the end posts. The actual number of web systems is usually from two to six or eight, but in some cases more are used, the number depending to a great extent on the depth of the portal. No definite rule can be stated for sketching out or selecting the arrangement of web members, which depends somewhat on the judgment of the designer, and is frequently controlled by esthetic considerations.

If the portal shown in Fig. 8 is considered cut by a plane aa' at right angles to the flanges AA' and BB' , and the portion on the left of the section is considered as a free body, it will be seen that the shear on the section and the moments about the points in the upper and lower flanges at a distance x from the left end are the same as those found for a plate-girder portal. They are, then, as follows:

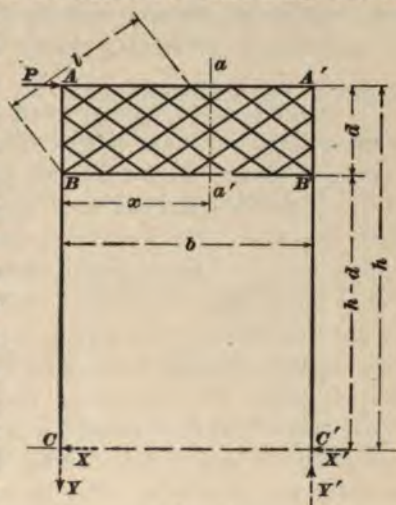


FIG. 8

$$\text{shear} = \frac{Ph}{b}; \text{ moment at } a = \frac{Ph}{2} - \frac{Phx}{b};$$

$$\text{moment at } a' = \frac{Ph}{2} + \frac{Pd}{2} - \frac{Phx}{b}$$

As in the case of the multiple-system truss, it is impossible to calculate the stresses in the members directly by the equations of equilibrium; in this case, it is also impossible to find what proportion of the load goes to each system, so that the

systems cannot be treated independently, as in the case of the truss. For purposes of calculation, however, it is customary to assume that the shear on any section cut by a plane at right angles to the flanges is evenly distributed among the web members cut by the plane; if the number of web members cut is n , the Y component in each member is assumed to be $\frac{Ph}{nb}$. This assumption is not absolutely

correct, but is probably as close to the true condition of affairs as any assumption that can be made. In the present case, where all the web members have the same inclination, if the length of one member between flanges is l , the stress in each diagonal is $\frac{Ph}{nb} \times \frac{l}{d}$. When the wind comes

from the left, the shear is negative, and causes tension in those members that slope downwards to the left, and compression in those that slope upwards to the left; when the wind comes from the right, the shear is positive, and the stresses are opposite to the above, but of the same numerical values.

The stresses in the flanges may be calculated most readily by the method of moments. As the Y components in the various web members are equal, and as the inclinations of the members are the same, their X components must be equal also. Then, if a plane is passed through the center of one of the panels of the flanges, it will cut the web members at their intersections. The lever arms, and, therefore, the moments of the Y components of the stresses in the web members, about the points a and a' in the flanges, will be equal to zero. At each intersection, there are two X components, equal and opposite; the sum of their moments about the points a and a' is, therefore, equal to zero. The moments of the external forces about a and a' have already been found. Then, the stress in AA' is equal to the moment at a' divided by d , and the stress in BB' is equal to the moment at a divided by d . The stress in AA' is

$$\frac{\frac{Ph}{2} + \frac{Pd}{2} - \frac{Phx}{b}}{d} = \frac{Ph}{2d} + \frac{P}{2} - \frac{Phx}{bd}$$

and the stress in BB' is

$$\frac{\frac{Ph}{2} - \frac{Phx}{b}}{d} = \frac{Ph}{2d} - \frac{Phx}{bd}$$

The actual stresses will depend on the value of x . If the web members divide the flanges into m panels of the same length p , the smallest value of x will be $\frac{p}{2}$, and the largest, $mp - \frac{p}{2}$, or $\frac{2m-1}{2}p$. When there is a large number of small panels, the smallest value of x may be taken equal to zero, and the largest equal to mp , or b . Then:

When $x = 0$,

stress in $AA' = \frac{Ph}{2d} + \frac{P}{2}$, compression at left end

When $x = b$,

stress in $AA' = -\frac{Ph}{2d} + \frac{P}{2}$, tension at right end

When $x = 0$,

stress in $BB' = \frac{Ph}{2d}$, tension at left end

When $x = b$,

stress in $BB' = -\frac{Ph}{2d}$, compression at right end

When the wind comes from the right, x is the distance from the right end, and the stresses in the members are reversed.

14. Portal With Knee Braces.—When the depth of the truss is such that there is insufficient room above the traffic for any of the portals that have been previously described, an arrangement similar to that shown in Fig. 4 (*d*) is frequently used. The horizontal member $ADD'A'$ is the portal and connects the end posts at the top; the inclined members, or knee braces, BD and $B'D'$ connect the end posts to the under side of the portal, which is usually a plate or lattice girder. The reactions at C and C' , and the stresses in the end posts below the points B and B' , are the same as found in Art. 10. The knee braces are subjected to direct stresses of tension and compression; the portal is subjected

to direct stresses of tension and compression, and also to shearing and bending stresses.

15. The stress in BD , Fig. 9 (a), may be found by taking moments about A of all the forces on the left of a plane of section pq cutting BD and AD . Denoting the X

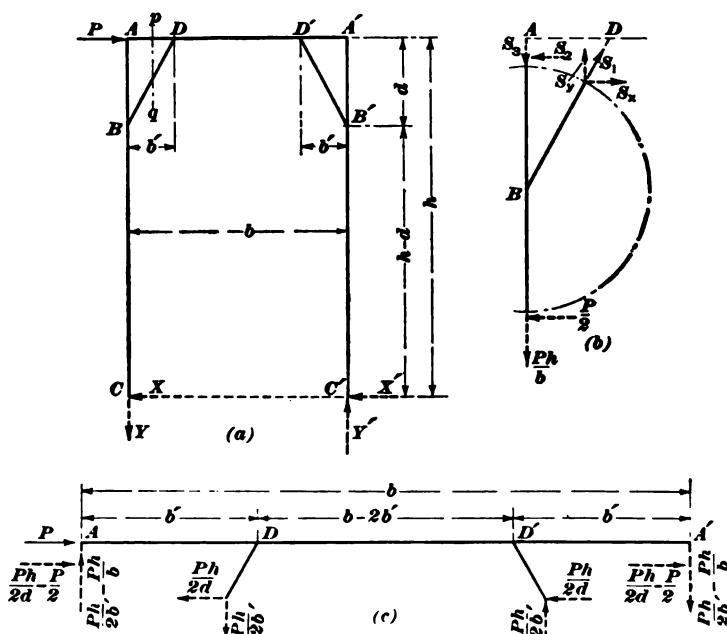


FIG. 9

component of the stress in BD by S_x , and the length of BD by l , and taking moments about A , we have

$$\Sigma M = \frac{P}{2} h - S_x d = 0;$$

whence
$$S_x = \frac{P h}{2 d}$$

and the stress in BD is

$$S_x \sec ADB = \frac{P h}{2 d} \times \frac{BD}{AD} = \frac{P h}{2 d} \times \frac{l}{b'}$$

Also,
$$S_y = \frac{P h}{2 b'}$$

Applying the equations $\Sigma X = 0$ and $\Sigma Y = 0$ to all the forces acting at the point B , Fig. 9 (*b*), the stresses in AB are found as follows:

$$\Sigma X = \frac{P}{2} - \frac{Ph}{2d} + S_s = 0;$$

whence $S_s = \frac{Ph}{2d} - \frac{P}{2}$, shear in AB .

$$\Sigma Y = \frac{Ph}{b} - \frac{Ph}{2b'} + S_s = 0;$$

whence $S_s = \frac{Ph}{2b'} - \frac{Ph}{b}$, compression in AB

In like manner, it may be shown that the stress in $B'D'$ is compression equal to $\frac{Ph}{2db'}$; and the stresses in $A'B'$ are shear equal to $\frac{Ph}{2d} - \frac{P}{2}$, and tension equal to $\frac{Ph}{2b'} - \frac{Ph}{b}$.

As this style of portal is usually very shallow and rests on top of the chords, it will be sufficiently accurate to ignore the depth of the portal itself in computing the moments.

The X and Y components of the forces that act on the portal are shown in Fig. 9 (*c*); the stresses will now be found.

At the joint A , the forces are as follows: the wind pressure P , the shear and direct stress in AB , and the stresses in AD . Denoting the direct stress in AD by S_s , and writing the equation $\Sigma X = 0$, we have

$$\Sigma X = S_s - P - \frac{Ph}{2d} + \frac{P}{2} = 0;$$

whence $S_s = \frac{Ph}{2d} + \frac{P}{2}$, compression

Denoting the shear in AD by S_s , and writing the equation $\Sigma Y = 0$, we have

$$\Sigma Y = S_s - \frac{Ph}{2b'} + \frac{Ph}{b} = 0;$$

whence $S_s = \frac{Ph}{2b'} - \frac{Ph}{b}$, positive shear

The bending moment at any section at a distance x to the right of A is

$$M = \left(\frac{Ph}{2b'} - \frac{Ph}{b} \right) x$$

At D , $x = b'$, and $M_D = \frac{Ph}{2} - \frac{Phb'}{b}$, positive moment.

The forces at the joint D are the stresses in AD , DD' , and BD . Denoting the direct stress in DD' by S_* , and writing the equation $\Sigma X = 0$, we have

$$\Sigma X = S_* - \left(\frac{Ph}{2d} + \frac{P}{2} \right) + \frac{Ph}{2d} = 0;$$

whence $S_* = \frac{P}{2}$, compression

Denoting the shear in DD' by S_r , and writing the equation $\Sigma Y = 0$,

$$\Sigma Y = S_r - \frac{Ph}{2b'} + \left(\frac{Ph'}{2b'} - \frac{Ph}{b} \right) = 0;$$

whence $S_r = \frac{Ph}{b}$, negative shear

The bending moment at any section of DD' , at a distance x_1 from A , is

$$\begin{aligned} M &= \left(\frac{Ph}{2b'} - \frac{Ph}{b} \right) x_1 - \frac{Ph}{2b'} (x_1 - b') \\ &= \frac{Ph}{2} - \frac{Phx_1}{b}, \text{ positive moment} \end{aligned}$$

At D' , $x_1 = b - b'$, and $M_{D'} = \frac{Phb'}{b} - \frac{Ph}{2}$, positive moment.

In like manner, the stresses in $D'A'$ may be found. They are as follows:

$$\frac{Ph}{2b'} - \frac{Ph}{b}, \text{ positive shear; and } \frac{Ph}{2d} - \frac{P}{2}, \text{ tension}$$

16. Portal With Curved Brackets. With the portals shown in Fig. 4 (*b*) and (*c*), the form of knee brace or bracket shown in Fig. 4 (*c*) is frequently used; it serves a double purpose in decreasing the bending on the end post and adding slightly to the architectural appearance of the entrance to the bridge. In this case, the maximum bending moment on the end post occurs at E and E' , and is equal to $\frac{P}{2}(h - d - d')$, which is $\frac{Pd'}{2}$ less than when no bracket is used.

The bending moment on the portal where it is in contact with the bracket is less than when no bracket is used. In

calculating the moments at various sections along the portal, the formulas already found in Arts. 12 and 13 may be used, x varying from a minimum value d' to a maximum value $b - d'$ for the plate girder, and from $d' + \frac{p}{2}$ to $b - d' - \frac{p}{2}$ for the lattice portal. The shear between the brackets will be the same as when no brackets are used.

17. Formulas for Portals.—The formulas necessary for the designs of the various portals that have been analyzed are shown in Fig. 10 (*a*) to (*e*), with the wind assumed as coming from the left. When the wind comes from the right, the formulas will be the same, but will in each case apply to the corresponding member on the other side on the bridge.

EXAMPLE.—In the portals shown in Fig. 4, let $h = 32$ feet, $d = 8$ feet, $b = 18$ feet, $b' = 4$ feet, $d' = 3$ feet, and $P = 24,000$ pounds, the wind coming from the left. What are the wind stresses in the different portals and end posts, using the formulas given in Fig. 10?

SOLUTION.—In Fig. 10 (*a*), (*b*), (*c*), and (*d*), the stresses in BC are as follows:

$$\text{The tension is } \frac{Ph}{b} = \frac{24,000 \times 32}{18} = 42,700 \text{ lb.}$$

$$\text{The shear is } \frac{P}{2} = 24,000 \div 2 = 12,000 \text{ lb.}$$

The bending moment at B is

$$\frac{P}{2}(h - d) = \frac{24,000}{2} \times (32 - 8) = 288,000 \text{ ft.-lb.}$$

The stresses in $B'C'$ are the same numerically as those in BC ; the direct stress of 42,700 lb. in $B'C'$ is compression.

In Fig. 10 (*e*), the stresses in EC and $E'C'$ are the same, respectively, as those in BC and $B'C'$, just given, except that the bending moment at E and at E' is equal to

$$\frac{P}{2}(h - d - d') = \frac{24,000}{2} \times (32 - 8 - 3) = 252,000 \text{ ft.-lb.}$$

1. The stresses in the members of the braced portal shown in Fig. 10 (*a*) will next be found.

The stresses in AA' , $A'B$, and BB' are as follows:

$$\text{in } AA', \quad \frac{Ph}{2d} + \frac{P}{2} = \frac{24,000 \times 32}{2 \times 8} + \frac{24,000}{2} = 60,000 \text{ ft.-lb.}$$

$$\text{in } A'B, \quad \frac{Phl}{db} = \frac{24,000 \times 32 \times \sqrt{18^2 + 8^2}}{8 \times 18} = 105,100 \text{ lb., tension}$$

$$\text{in } BB', \quad \frac{Ph}{2d} = \frac{24,000 \times 32}{2 \times 8} = 48,000 \text{ lb., compression}$$

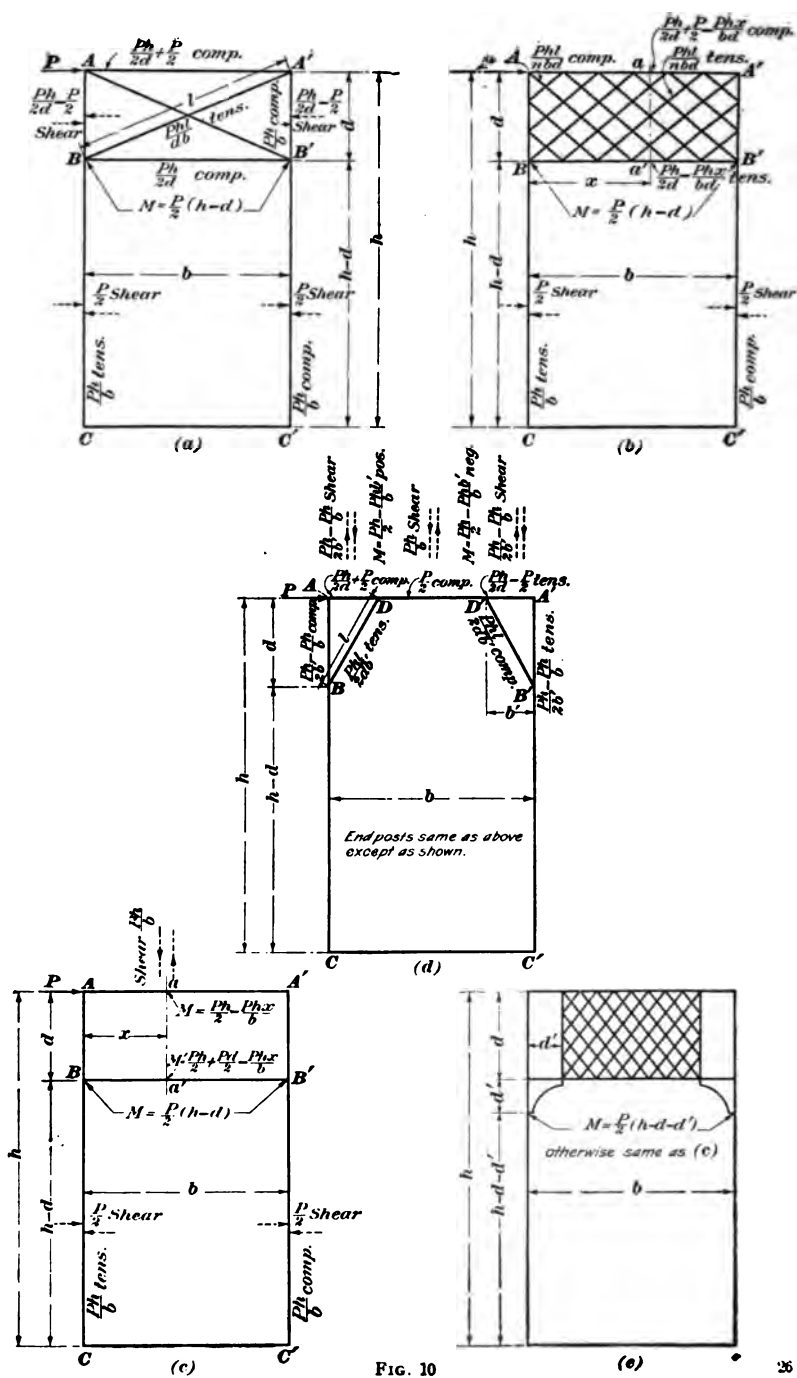


FIG. 10

The direct stress in AB is 0.

The shear in AB and $A'B'$ is

$$\frac{Ph}{2d} - \frac{P}{2} = \frac{24,000 \times 32}{2 \times 8} - \frac{24,000}{2} = 36,000 \text{ lb.}$$

The direct stress in $A'B'$ is

$$\frac{Ph}{b} = \frac{24,000 \times 32}{18} = 42,700 \text{ lb., compression}$$

2. The stresses in the members of the lattice portal shown in Fig. 10 (b) will next be found.

The stress in each web member is equal to

$$\frac{Phl}{nb d} = \frac{24,000 \times 32 \times \sqrt{9^2 + 8^2}}{6 \times 18 \times 8} = 10,700 \text{ lb.,}$$

tension in those members that slope upwards to the right, and compression in the remaining members.

The stress in AA' is equal to

$$\begin{aligned} \frac{Ph}{2d} + \frac{P}{2} - \frac{Phx}{bd} &= \frac{24,000 \times 32}{2 \times 8} + \frac{24,000}{2} - \frac{24,000 \times 32 \times x}{8 \times 18} \\ &= 60,000 - \frac{16,000}{3}x \end{aligned}$$

Since there are six panels and $b = 18$ ft., $p = \frac{18}{6} = 3$ ft.; then, at the left end $x = 1.5$ ft. and the stress in AA' is

$$60,000 - \frac{16,000}{3} \times 1.5 = 52,000 \text{ lb., compression}$$

At the right end, $x = 16.5$ ft., and the stress in AA' is

$$60,000 - \frac{16,000}{3} \times 16.5 = -20,000 \text{ lb., tension}$$

The stress in BB' is

$$\frac{Ph}{2d} - \frac{Phx}{bd} = \frac{24,000 \times 32}{2 \times 8} - \frac{24,000 \times 32 \times x}{8 \times 18} = 48,000 - \frac{16,000}{3}x$$

At the left end, the stress in BB' is

$$48,000 - \frac{16,000}{3} \times 1.5 = 40,000 \text{ lb., tension}$$

At the right end, the stress in BB' is

$$48,000 - \frac{16,000}{3} \times 16.5 = -40,000 \text{ lb., compression}$$

3. The shears and moments on the plate-girder portal shown in Fig. 10 (c) will next be found.

Shear at any section is $\frac{Ph}{b} = 42,700$ lb., negative shear.

The moments in the upper flange are: at the left end, $M = \frac{Ph}{2} = 384,000$ ft.-lb.; and, at the right end, $M = -\frac{Ph}{2} = -384,000$ ft.-lb.

The moments in the lower flange are: at the left end,

$$M' = \frac{Ph}{2} + \frac{Pd}{2} = 480,000 \text{ ft.-lb.}$$

and, at the right end,

$$M' = \frac{Pd}{2} - \frac{Ph}{2} = -288,000 \text{ ft.-lb.}$$

4. The stresses in the portal with knee braces, shown in Fig. 10 (d), will next be found.

The stresses in BD and $B'D'$ are as follows:

$$\text{in } BD, \frac{Phl}{2db'} = \frac{24,000 \times 32 \times \sqrt{8^2 + 4^2}}{2 \times 8 \times 4} = 107,300 \text{ lb., tension}$$

$$\text{in } B'D', \frac{Phl}{2db'} = 107,300 \text{ lb., compression}$$

The direct stresses in AB and $A'B'$ are:

in AB ,

$$\frac{Ph}{2b'} - \frac{Ph}{b} = \frac{24,000 \times 32}{2 \times 4} - \frac{24,000 \times 32}{18} = 53,300 \text{ lb., compression}$$

in $A'B'$,

$$\frac{Ph}{2b'} - \frac{Ph}{b} = 53,300 \text{ lb., tension}$$

The direct stresses in AD , DD' , and $D'A'$ are:

$$\text{in } AD, \frac{Ph}{2d} + \frac{P}{2} = \frac{24,000 \times 32}{2 \times 8} + \frac{24,000}{2} = 60,000 \text{ lb., compression}$$

$$\text{in } DD', \frac{P}{2} = \frac{24,000}{2} = 12,000 \text{ lb., compression}$$

$$\text{in } D'A', \frac{Ph}{2d} - \frac{P}{2} = \frac{24,000 \times 32}{2 \times 8} - \frac{24,000}{2} = 36,000 \text{ lb., tension}$$

The shears in AD , DD' , and $D'A'$ are as follows:

$$\text{in } AD, \frac{Ph}{2b'} - \frac{Ph}{b} = \frac{24,000 \times 32}{2 \times 4} - \frac{24,000 \times 32}{18} = 53,300 \text{ lb.}$$

$$\text{in } DD', \frac{Ph}{b} = \frac{24,000 \times 32}{18} = 42,700 \text{ lb.}$$

$$\text{in } D'A', \frac{Ph}{2b'} - \frac{Ph}{b} = 53,300 \text{ lb.}$$

The bending moments at D and D' are:

$$\text{at } D, \frac{Ph}{2} - \frac{Phb'}{b} = \frac{24,000 \times 32}{2} - \frac{24,000 \times 32 \times 4}{18} = 213,300 \text{ ft.-lb.}$$

$$\text{at } D', \frac{Ph}{2} - \frac{Phb'}{b} = 213,300 \text{ ft.-lb.}$$

5. The stresses in the portal with curved brackets will next be found.

The stresses in the web members are the same as found in 2. The stresses in the flanges are different, however, from those there found, because the smallest value of x is 4.5 ft. and the largest is 13.5 ft.

The stresses in AA' and BB' are as follows:

in AA' (at left end),

$$60,000 - \frac{16,000}{3} \times 4.5 = 36,000 \text{ lb., compression}$$

in AA' (at right end),

$$60,000 - \frac{16,000}{3} \times 13.5 = -12,000 \text{ lb., tension}$$

in BB' (at left end),

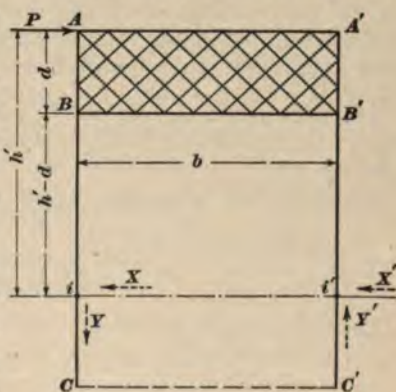
$$48,000 - \frac{16,000}{3} \times 4.5 = 24,000 \text{ lb., tension}$$

in BB' (at right end),

$$48,000 - \frac{16,000}{3} \times 13.5 = -24,000 \text{ lb., compression}$$

18. Fixed End Posts.—In the preceding discussion, it was assumed that the end posts were hinged at the bottom, or free to turn. When they simply rest on pedestals, this is very near the actual case, and the bending moment at the lower joint is zero. When they are firmly riveted at their lower ends to floorbeams or cross-braces of considerable depth, they offer some resistance to bending at these points, and this bending is opposite in direction to that at the lower connection of the portal.

Then there is a point of inflection somewhere between the bottom of the end post and the portal, the actual location of which depends on the nature of the top and bottom connections. For all ordinary cases, in a well-designed bridge, if the end posts are fixed at top and bottom, the point of inflection will be about midway between the



lower joint and the lower line of the portal; to allow for lack of sufficient rigidity at the lower joint, it is well to consider the point of inflection never higher than one-half the distance from the lower joint to the lower line of the portal. The actual location of the point of inflection may be found for each case, but as its determination involves the use of advanced mathematics, it will not be given here.

As the bending moment at the points of inflection is zero, the only stresses in the end posts at those points are the direct stresses and shears, the Y and X components. In Fig. 11, let i and i' represent these points at a distance h' from the top of the portal. Then, if moments are taken about i and i' , it will be found that

$$Y = P \frac{h'}{b} = Y'; \text{ also, } X = \frac{P}{2}$$

As these are the same as though the end posts were supported at the points i and i' , the stresses in the portals may be found by means of the formulas already deduced, substituting for h , in each case, the distance h' . It is clear that the stresses in the end posts and portal are decreased by fixing the end posts at the bottom.

EXAMPLE.—In Fig. 4 (*b*), suppose that the wind pressure and dimensions are the same as given in the example in Art. 17. What are the wind stresses in the end posts and members of the portal, assuming that the point of inflection is located one-third of the distance from C to B ; that is, 8 feet above C and C' , so that $h' = 24$ feet?

SOLUTION.—The stresses will be found by the formulas given in Fig. 10 (*b*), substituting h' for h . Then, the direct stresses in the end posts are

$$\frac{Ph'}{b} = \frac{24,000 \times 24}{18} = 32,000 \text{ lb., tension in } BC \text{ and compression in } B'C$$

The shear in BC and $B'C'$ is

$$\frac{P}{2} = \frac{24,000}{2} = 12,000 \text{ lb.}$$

The moment at B and B' is

$$\frac{P}{2} (h' - d) = 12,000 \times 16 = 192,000 \text{ ft.-lb.}$$

The stress in each web member is

$$\frac{Ph'l}{nbd} = \frac{24,000 \times 24 \times \sqrt{9^2 + 8^2}}{6 \times 18 \times 8} = 8,000 \text{ lb.,}$$

tension in those members that slope upwards to the right and compression in the others.

The stress in AA' is

$$\frac{Ph'}{2d} + \frac{P}{2} - \frac{Ph'x}{bd} = \frac{24,000 \times 24}{2 \times 8} + 12,000 - \frac{24,000 \times 24 \times x}{8 \times 18} = 48,000 - 4,000x$$

At the left end, $x = 1.5$, and the stress is

$$48,000 - 4,000 \times 1.5 = 42,000 \text{ lb., compression}$$

At the right end, $x = 16.5$, and the stress is

$$48,000 - 4,000 \times 16.5 = -18,000 \text{ lb., tension}$$

The stress in BB' is

$$\frac{Ph'}{2d} - \frac{Ph'x}{bd} = \frac{24,000 \times 24}{2 \times 8} - \frac{24,000 \times 24 \times x}{8 \times 18} = 36,000 - 4,000x$$

At the left end, $x = 1.5$, and the stress is

$$36,000 - 4,000 \times 1.5 = 30,000 \text{ lb., tension}$$

At the right end, $x = 16.5$, and the stress is

$$36,000 - 4,000 \times 16.5 = -30,000 \text{ lb., compression}$$

It will be seen that in this example all the stresses are less than those found in the example in Art. 17, except the shear in the end posts, which is the same.

19. Wind Effects on the Main Truss.—The Y components of the stresses in the end posts act directly along the members, and must be combined with the dead- and live-load stresses, in order to get the maximum stresses. On the leeward side, the compression due to the wind must be added to the dead- and live-load compression to get the maximum; on the windward side, the tension due to the wind must be subtracted from the dead- and live-load compression, or probably from the dead-load compression, to get the minimum stress. The latter is of interest only when the tension due to wind is greater than the dead-load compression, in which case the truss must be well bolted down to resist overturning. The direct stress in an inclined end post due to the wind causes a stress in the lower chord, as shown in Fig. 12. If Y is the direct wind stress in the end post, then $Y \cos H$ is the wind stress in the lower chord, due to that in the end post, and it is constant from end to end, being tension on the leeward side, and compression on the windward.

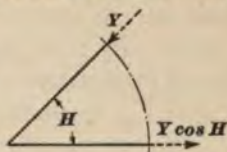


FIG. 12

The total wind effects on the chords of the main trusses are as follows:

Top Chords.—Stresses due to their positions as chord members of the upper lateral truss, and stresses caused by the increase or decrease in the vertical load due to the overturning effect.

Bottom Chords.—Stresses due to their positions as chord members of the lower lateral truss, stresses caused by the increase or decrease in the vertical loads due to the overturning effect, and stresses due to the direct wind stresses in the inclined end posts.

20. Sway Frames.—In a deck bridge, the wind pressure that comes to the upper lateral system is transmitted to the abutments or supports by bracing in the planes of the end

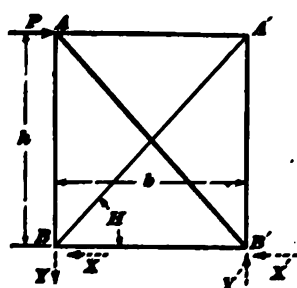


FIG. 13

posts, similar to that shown in Fig. 13. The two transverse members AA' and BB' , together with the diagonals AB and $A'B'$, are usually called a sway frame. The diagonals are assumed to resist tension only; so, if the wind comes from the left, the member $A'B$ will be in action.

The reactions at B and B' , due to the load P at A , may be found from the equations of equilibrium. Taking moments about B and B' ,

$$Y = P \frac{h}{b}, \text{ and } Y' = P \frac{h}{b} = Y$$

In regard to the X component of the reactions, it is evident that there is some at B and some at B' ; but, as the point of overturning is at B' , it is safe to assume $X' = P$ and $X = 0$. Then, the stresses in the members are as follows:

MEMBER	STRESS
AA'	P , compression
$A'B$	$P \sec H$, tension
AB'	0
$A'B'$	$P \frac{h}{b}$, compression
AB	0
BB'	P , compression

21. Deck and through bridges are given additional lateral stiffness by means of vertical transverse or sway frames at every panel point or at every other panel point. In the through bridge, these are of the same general form as the

portals already discussed; they are made as deep as conditions will allow. In the deck bridge, they are made the same depth as the trusses. As the upper and lower lateral trusses, together with the bracing in the planes of the end posts, form a complete system for transmitting lateral forces to the supports, it is impossible to tell how much load comes on these intermediate frames. The stresses in them are usually found in the same way as in the end sway frames and portals already described, on the assumption that they are supported at the bottom chord, the load at the top joint being taken equal to one-half a top panel load. This simply affords a basis for design, and is not intended to relieve the wind pressure on the upper lateral truss.

EXAMPLE.—In Fig. 13, let $h = 25$ feet and $b = 18$ feet. If the wind pressure P is equal to 20,000 pounds coming from the left, what are the stresses in AA' , $A'B$, and BB' ?

SOLUTION.—The stress in AA' is $P = 20,000$ lb., compression; the stress in $A'B$ is

$$P \sec H = 20,000 \times \frac{\sqrt{25^2 + 18^2}}{18} = 34,200 \text{ lb., tension;}$$

and the stress in BB' is $P = 20,000$ lb., compression. Also, the compression in $A'B'$ is

$$P \frac{h}{b} = 20,000 \times \frac{25}{18} = 27,800 \text{ lb. Ans.}$$

EXAMPLES FOR PRACTICE

1. In the portal represented in Fig. 4 (*a*), if $h = 32$ feet, $b = 16$ feet, and $P = 20,000$ pounds coming from the left, what is the direct stress in BC , assuming the end posts to be hinged at the bottom?

Ans. 40,000 lb., tension

2. If the dimensions and wind pressure are the same as in the preceding example, and $d = 4$ feet, what is the stress in BB' ?

Ans. 80,000 lb., compression

3. If the dimensions and wind pressure on the portal shown in Fig. 4 (*b*) are the same as described in examples 1 and 2, what is the stress in the first panel at the left end of AA' ?

Ans. 76,700 lb., compression

4. In the portal shown in Fig. 4 (*d*), if $h = 24$ feet, $b = 18$ feet, $d = 8$ feet, $b' = 6$ feet, and $P = 12,000$ pounds, coming from the right, what is the stress in BD ?

Ans. 30,000 lb., compression

CENTRIFUGAL FORCE

22. Value of Centrifugal Force.—When the track that crosses a bridge is curved, the cars tend to move along the tangent, thereby exerting a lateral thrust, the amount of which can be found from the formula for centrifugal force given in *Kinematics and Kinetics*; namely,

$$F = \frac{Wv^2}{gr}$$

In the present case, W is the weight of the train, in pounds per linear foot; v , the velocity of the train, in feet per second; g , the acceleration due to gravity; r , the radius of the curve, in feet; and F , the value of the centrifugal force, in pounds per linear foot. In practice, it is customary to express the speed of the train in miles per hour, and the degree of curve in degrees. If V represents the speed in miles per hour, then

$$v = \frac{5,280 V}{60 \times 60} \text{ feet per second}$$

If D represents the degree of curve, the radius r is approximately equal to $\frac{5,730}{D}$ feet. Substituting these values in the formula for F , we get

$$F = \frac{W \times \left(\frac{5,280 V}{60 \times 60} \right)^2}{32.16 \times \frac{5,730}{D}} = .00001167 V^2 D W$$

The value of the centrifugal force is sometimes given as a percentage of the live load. For instance, for a speed of 50 miles per hour on a curve of 2° , the centrifugal force is $.00001167 \times 50^2 \times 2 \times W = .0584 W$;

that is, 5.84 per cent. of the live load. If, in this case, W is 4,000 pounds per linear foot, and F is $4,000 \times .0584 = 234$ pounds.

23. Distribution of Load.—The track on a curve is subject to the action of the weight and centrifugal force of the train. In practice, it is customary to make the outer rail on a curve higher than the inner rail, by an amount sufficient

to cause the resultant of the weight and centrifugal force to be perpendicular to the plane of the track half way between the rails. Under this condition, the loads that come on the rails are equal; but, as the track is curved, and, therefore, not the same distance from the trusses at every point, the loads at the panel points are not equal. Fig. 14 represents the cross-section of a through bridge having the width b , and shows the center line cc' of the bridge; the surface TT' of the track; the plane aa' of the lower lateral truss; the line of action, perpendicular to TT' , of the resultant Q of the weight and the centrifugal force; and the intersections p and q , respectively, of the line of action of Q with TT' and aa' . The inclination of TT' is much exaggerated to make the explanation clearer.

If Q is resolved at q into its vertical and horizontal components W and F , respectively, the force F , being horizontal, will be resisted directly by the lower lateral truss, and W will be resisted by the vertical trusses. If e' is the distance of q from the center line of the bridge, the amount of W that goes to the outside truss is

$$\frac{W\left(\frac{b}{2} + e'\right)}{b}, \text{ or } \frac{W}{2} + W\frac{e'}{b}$$

and the amount that goes to the inside truss is

$$\frac{W\left(\frac{b}{2} - e'\right)}{b}, \text{ or } \frac{W}{2} - W\frac{e'}{b}$$

24. Eccentricity of Track.—The distance e , Fig. 14, from the center line of the track to the center line of the bridge is called the **eccentricity** of the track. The eccentricity varies along the bridge, as shown in Fig. 15, in which LM is the center line of the bridge; ABC , the center line of the curved track; and $AB'C$, the chord to the curve between the ends of the bridge, BB' being the middle ordinate. The values of the eccentricity for the different panel points can be computed from the curve of the track; they may be called e_1, e_2, e_3 , etc. The distance e' , Fig. 14, is the sum of e and e'' ,

and the amount that goes to the inside truss is

$$\frac{W}{2} + W \frac{e'}{b}$$

For spans shorter than about 75 feet, it is customary to make the distance BB'' , Fig. 15, from the center line of the track to the center line of the bridge, from one-half to one-third the middle ordinate BB' , and to assume that each girder carries one-half the vertical load, the same as for a bridge on a straight track. For longer spans, it is customary to find first the panel loads as though the track were straight, and then increase or decrease them according to the location of the center line of the track at each floorbeam.

25. Lateral Bracing.—In a through bridge, the centrifugal force is transmitted to the lower lateral truss by the floor, and the stresses in the members are found in the same way as those due to wind. In a deck bridge, the centrifugal force is transmitted by the floor to the upper lateral truss, and by it to the sway frames; the stresses in the members are found in the same way as those due to wind. Centrifugal force causes stresses in but one set of diagonals, as it acts in but one direction.

EXAMPLE.—A train of cars weighing 5,000 pounds per linear foot, moving over a curve of 3° at the rate of 40 miles per hour, crosses an eight-panel through bridge having a span of 160 feet and a width of 20 feet. If the outer rail is elevated .265 ft., and the center line of the bridge is located two-thirds of the middle ordinate from the chord to the curve ($BB'' = \frac{1}{3} BB'$, Fig. 15), what is: (a) the panel load

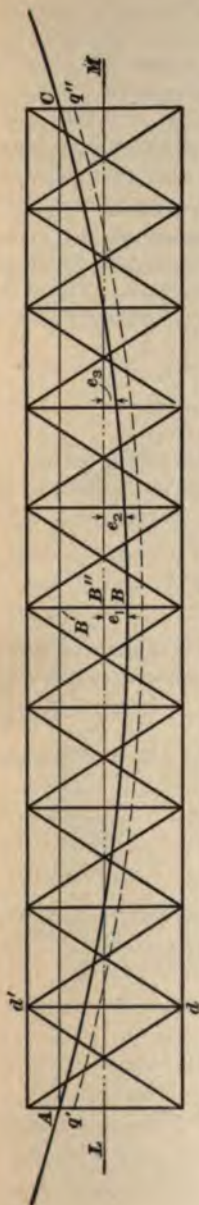


FIG. 15

for the lower lateral truss caused by the centrifugal force; (b) the correction to be applied at each panel point of the outside truss due to the eccentricity? Assume that the distance from the surface of the track to the lower lateral truss is 4 feet, and that the gauge of the track is 4.71 feet.

SOLUTION.—(a) $F = .00001167 \times 40 \times 40 \times 3 \times 5,000 = .056 \times 5,000 = 280$ lb. per lin. ft.

The panel load for the lateral truss is $280 \times 20 = 5,600$ lb. Ans.

(b) The perpendicular distance from the center of the curve ($R = 1,910.08$) to the chord (length = 160 ft.) is equal to 1,908.40 ft. Then, the middle ordinate is $1,910.08 - 1,908.40 = 1.68$ ft. The ordinates e_1 , e_2 , and e_3 at the other panel points, from the chord to the curve are as follows:

$$\text{Point } b, \quad e_1 = \sqrt{1,910.08^2 - 60^2} - 1,908.40 = .74 \text{ ft.}$$

$$\text{Point } c, \quad e_2 = \sqrt{1,910.08^2 - 40^2} - 1,908.40 = 1.26 \text{ ft.}$$

$$\text{Point } d, \quad e_3 = \sqrt{1,910.08^2 - 20^2} - 1,908.40 = 1.58 \text{ ft.}$$

The center line of the bridge is two-thirds the middle ordinate, or 1.12 feet, from the chord; then, the distances e from the center line of the bridge to the curve are as follows:

$$\text{Point } b, \quad e = .74 - 1.12 = -.38 \text{ ft.}$$

$$\text{Point } c, \quad e = 1.26 - 1.12 = .14 \text{ ft.}$$

$$\text{Point } d, \quad e = 1.58 - 1.12 = .46 \text{ ft.}$$

$$\text{Point } e, \quad e = 1.68 - 1.12 = .56 \text{ ft.}$$

The distance e'' from the center line of the track to the line of intersection of the resultant with the plane of the lateral system is equal to

$$\frac{4}{4.71} \times .265 = .22 \text{ ft.}$$

and to find the distances e' , it is simply necessary to add this distance to the values of e just found; they are as follows:

$$\text{Point } b, \quad e' = -.38 + .22 = -.16 \text{ ft.}$$

$$\text{Point } c, \quad e' = +.14 + .22 = +.36 \text{ ft.}$$

$$\text{Point } d, \quad e' = +.46 + .22 = +.68 \text{ ft.}$$

$$\text{Point } e, \quad e' = +.56 + .22 = +.78 \text{ ft.}$$

Then, the corrections at the panel points are as follows:

$$\text{Point } b, \quad W = (5,000 \times 20) \times \frac{-.16}{20} = -800 \text{ lb.}$$

$$\text{Point } c, \quad W = (5,000 \times 20) \times \frac{.36}{20} = +1,800 \text{ lb.}$$

$$\text{Point } d, \quad W = (5,000 \times 20) \times \frac{.68}{20} = +3,400 \text{ lb.}$$

$$\text{Point } e, \quad W = (5,000 \times 20) \times \frac{.78}{20} = +3,900 \text{ lb.}$$

EXAMPLES FOR PRACTICE

1. A railroad train weighing 5,000 pounds per linear foot is moving at the rate of 50 miles an hour on a 2° curve, and crosses a ten-panel through bridge having a span length of 200 feet. What is the panel load for the lower lateral truss due to the centrifugal force?

Ans. 5,835 lb.

2. A railroad train weighing 4,500 pounds per linear foot is moving at the rate of 60 miles an hour on a 3° curve, and crosses a twelve-panel bridge having a span length of 180 feet. What is the panel load for the lower lateral truss due to the centrifugal force?

Ans. 8,507 lb.

SKEW BRIDGES

26. Description.—In all that precedes, it has been assumed that the trusses were symmetrical about the center, and that the line connecting the end joints over the supports was at right angles to the center line of the bridge. It frequently happens, however, that the center line is not at right angles to the abutments or piers, in which case the line connecting the ends of the trusses is not at right angles to the center line. Such bridges are termed **skew bridges**, and give rise to conditions that require separate consideration.

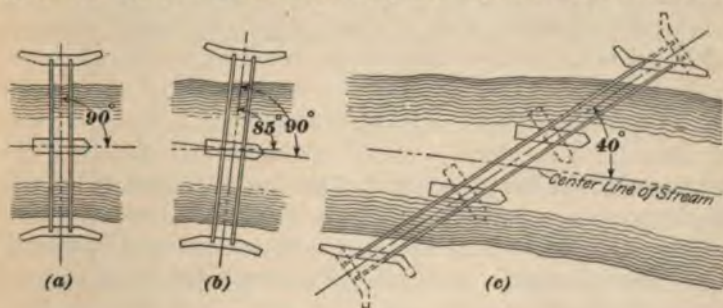


FIG. 16

27. Skew Crossings.—In locating a new line, either for a highway or for a railroad, it is sought, as far as possible, so to arrange the alinement that all bridge crossings shall be at right angles to the obstacle, usually either a road or a stream, to be crossed, as shown in Fig. 16 (a);

such a crossing is economical because it is shorter than any other. Local conditions and the topography of the country occasionally render this impossible, and the line crosses at an angle other than 90° . If the angle is only slightly less, as shown in Fig. 16 (*b*), the piers and abutments may be placed at right angles to the center line of the bridge; if much less, this plan may seriously interfere with the waterway or other clearance required, and increase the length of the bridge, as shown by the dotted lines in Fig. 16 (*c*). In such a case, the piers and abutments may be placed parallel to the stream or road, thereby making the same angle as the latter with the center line of the bridge. In eliminating skew grade crossings of two existing roads, it is frequently impossible to change the alinement of either and inadvisable to increase the length of the bridge by placing the abutments at right angles to the center line, in which case the latter may be placed parallel to the lower road, and the bridge built on a skew.

28. Arrangement of Panels, Floor, and Lateral System.—Near the ends of a skew bridge, the panel lengths are, in some cases, not equal to those near the center. This is due to the fact that the floorbeams are placed at right angles to the center line, and are spaced the same distance apart near the center of the bridge, bringing the panel points of the two trusses directly opposite; the amount of *skew* (that is, the distance, measured along the center line, that one truss is ahead of the other) is made up in the end panels. The arrangement of the panels and of the floor and lateral system depends to a great extent on the distance center to center of the trusses and on the angle of skew, each case requiring separate treatment. The general method will be illustrated by considering a special case.

In Fig. 17, (*a*) is the top view; (*b*), the side elevation of one truss; and (*c*), the lower lateral bracing of a skew bridge; the span of the bridge is equal to l , the perpendicular distance from center to center of the trusses is b , the angle between the center line of the bridge and the

abutments is B , and the height of trusses is h . In this particular case, the angle B of skew is about 45° , and the amount of skew s , or $h \cot B$, is very nearly equal to the panel length. Therefore, the hip vertical at one end of each truss may be located directly opposite the end joint of the other, making the end panels $f'e'$ and $f'e$, Fig. 17 (c), equal to s . In order that the end posts may have the same slope, it will be well to make the end panels $a'g'$ and ag at the other ends of the trusses equal

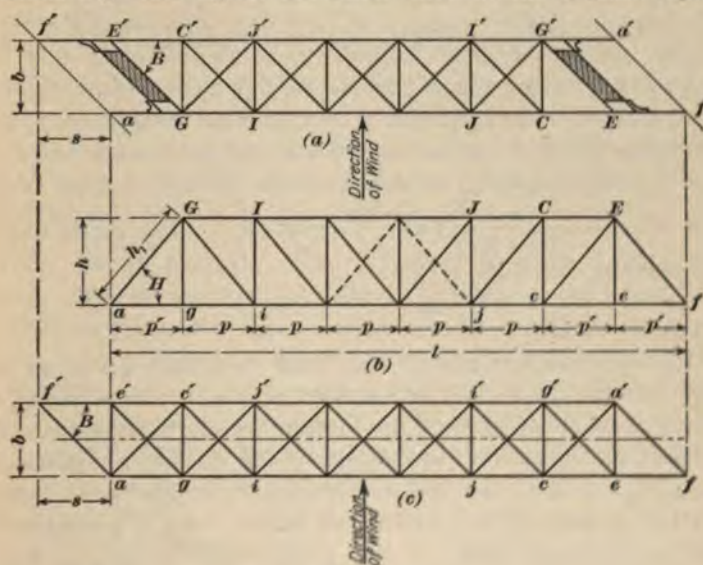


FIG. 17

also to s ; then, as the floorbeams are at right angles to the trusses, c and c' are, respectively, opposite g' and g , and the panels $c'e'$ and ec are also equal to s . For the sake of uniformity, each of these panel lengths, although equal to s , will be called p' . For each truss, then, there are three equal panels p' ; the remaining length, $l - 3p'$, included in the area $c'g'cg'$, Fig. 17 (c), may be divided into a number of equal panels—in this case, five.

29. Panel Loads and Reactions.—As the panel lengths are different, the panel loads will be different; those

from g to c , Fig. 17 (b), are found by multiplying the load per linear foot for one truss by one-half the sum of the lengths of the adjacent panels. The panel load at e is greatest when the truss is loaded from c to f , and is then equal to one-quarter the load in the area $cg'a'e$, plus the amount that comes from the triangular area $a'ef$, Fig. 17 (c). If the load is w pounds per square foot, uniformly distributed, the total load in the area $cg'a'e$ is $w b p'$, and the amount of this that goes to e is $\frac{w b p'}{4}$. As the area of the triangle $a'ef$ is $\frac{b p'}{2}$, the total load in this area is $\frac{w b p'}{2}$. To find what portion of this goes to e , it is well to consider it concentrated at the center of gravity of the triangle $a'ef$, at a distance from $a'f$ equal to one-third the distance from e to $a'f$. Then, taking moments about the line $a'f$, we find that one-third of $\frac{w b p'}{2}$, or $\frac{w b p'}{6}$, goes to e . The panel load at e is, then,

$$\frac{w b p'}{4} + \frac{w b p'}{6} = \frac{5 w b p'}{12}$$

In a railroad bridge, the live load is usually given as a uniform amount w'' per linear foot of track, and it is sufficiently accurate to assume the load in the end panel equal to the load per linear foot multiplied by the distance, measured along the center line of the bridge, from the end floorbeam $a'e$, Fig. 17 (c), to the abutment. As this distance is $\frac{p'}{2}$, the load is $\frac{w'' p'}{2}$; one-half of this, or $\frac{w'' p'}{4}$, goes to the end floorbeam, and one-half of this portion, or $\frac{w'' p'}{8}$, to e .

As the panel lengths are unequal, they cannot be used as a unit in calculating reactions and bending moments, as heretofore.

30. Lower Lateral Truss.—The floorbeams $a'e$ and $a'f$, Fig. 17 (c), rest on the masonry, or are connected to the trusses at a and a' , respectively, and it may be assumed that the lower lateral truss is composed of seven

panels with a span equal to $l - p'$. When the wind is blowing in the direction shown, the diagonal ac' transmits the pressure directly to the support at a , and $g'e$ to the end of the floorbeam ea' , which transmits the load that comes from $g'e$, and the wind panel load at e , to the support. As the panel lengths are unequal, the wind and centrifugal panel loads will be unequal.

31. Upper Lateral Truss.—In the upper lateral system, the transverse struts $C'G$ and CG' , Fig. 17 (*a*), connect the hip joint of one truss with the opposite joint of the other, and may be looked on as the limit of the upper lateral truss, which will then have five panels with a span equal to $l - 3p'$. When the wind is blowing in the direction shown, the diagonal GJ' transmits the pressure to the portal at G , and $I'C$ to the strut CG' at C , which transmits it, together with the wind panel load at C , to the portal at G' .

32. Skew Portal.—The portal of a skew bridge is called a **skew portal**. In Fig. 17 (*a*), the upper line GE' or $G'E$ of the portal lies in the plane of the upper lateral truss, and connects the upper joints of the end posts; it is horizontal, and makes the same angle B as the abutments with the center line of bridge. Each portal lies in a plane that passes through both end posts, the lower line being parallel to the upper line of the portal; the plane at the left end intersects the plane of the lower lateral bracing in the line af' , which is horizontal. The portal is shown revolved about the line af' into the plane of the paper and in an upright position in Fig. 18. The general methods that have been described for finding the stresses in portals may be applied in this case; as the calculation is somewhat complicated on account of the several angles involved, the formulas for the skew portal will now be found, considering the left portal and assuming the wind blowing in the direction shown. The wind pressure that is to be resisted by the portal is the amount transmitted to joint G , Fig. 17 (*a*), by the diagonal GJ' , plus the wind panel loads at G , C' , and E' . If the force acting at G at right angles to the upper chord

(parallel to the direction of the wind), due to the tension in GJ' and the wind panel loads at G and C' , is denoted by P , the force P_1 in the direction GE' is equal to $P \csc B$; if the wind panel load at E' is P' , then the force P_1' in the direction GE' is equal to $P' \csc B$, as shown in Fig. 18.

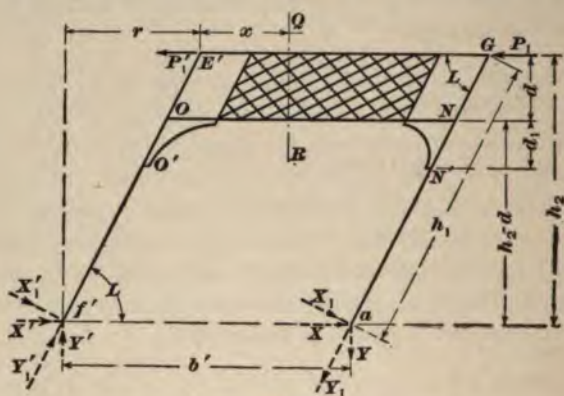


FIG. 18

The reactions at a and f' , resolved into components parallel and perpendicular, respectively, to the line of action of P_1 and P_1' , are X and X' , Y and Y' . Then,

$$P_1 + P_1' = X + X',$$

and, as X and X' may be assumed equal,

$$X = X' = \frac{P_1 + P_1'}{2}$$

Also, $Y = Y' = (P_1 + P_1') \frac{h_2}{b'}$

$$b' = b \csc B; h_2 = h_1 \sin L = h \csc H \sin L$$

The customary method of finding the stresses in the web members is to consider the portal cut by a plane, such as QR , Fig. 18, at right angles to the flanges, and assume that the shear on the plane is evenly distributed among the web members cut by it. If there are n members cut by the plane, the Y component in each member is $\frac{Y}{n}$, or, in this case, $(P_1 + P_1') \frac{h_2}{b' n}$; and the stress in the member is $\frac{Y}{n}$

multiplied by the cosecant of the angle between the web member and the flanges.

The stresses in the flanges may be found approximately in the same way as in Art. 13; that is, by considering the portal cut by a plane at right angles to the flanges, treating the portion on one side of the plane as a free body, and applying the equation $\Sigma M = 0$ to this portion, neglecting the effect of the web. In Fig. 18, let QR be at a distance x from E' , and assume that the stress in GE' is compression, and in NO tension. Then, applying the equation $\Sigma M = 0$ to the portion of the portal on the left of QR , the stresses in GE' and NO are as follows:

$$\text{in } GE', \quad \frac{Y'(r+x) - X'(h_2 - d) - P'_1 d}{d}, \text{ compression;}$$

$$\text{in } NO, \quad \frac{Y'(r+x) - X' h_2}{d}, \text{ tension}$$

The value of r is found by the equation

$$r = h_1 \cos L$$

In case the numerical values of these stresses come out negative, they are opposite in character to the stresses just given; that is, the stress in GE' will be tension, and in NO compression.

In order to find the stresses in the end posts below the portal, it is necessary to compute the components X_1 and X'_1 , Y_1 and Y'_1 of X and X' , Y and Y' , in the direction of, and at right angles to, the end posts. Letting L represent the angle $E' f' a$, the components are as follows:

at f' ,

$$X'_1 = X' \sin L - Y' \cos L; \quad Y'_1 = Y' \sin L + X' \cos L;$$

and at a ,

$$X_1 = X \sin L + Y \cos L; \quad Y_1 = Y \sin L - X \cos L$$

Then, the stresses in the end posts are as follows:

The stress in $f' O'$ is Y'_1 , compression.

The shear in $f' O'$ is X'_1 .

The moment at O' is $X'_1 [(h_2 - d - d_1) \csc L]$

The stress in $a N'$ is Y_1 , tension.

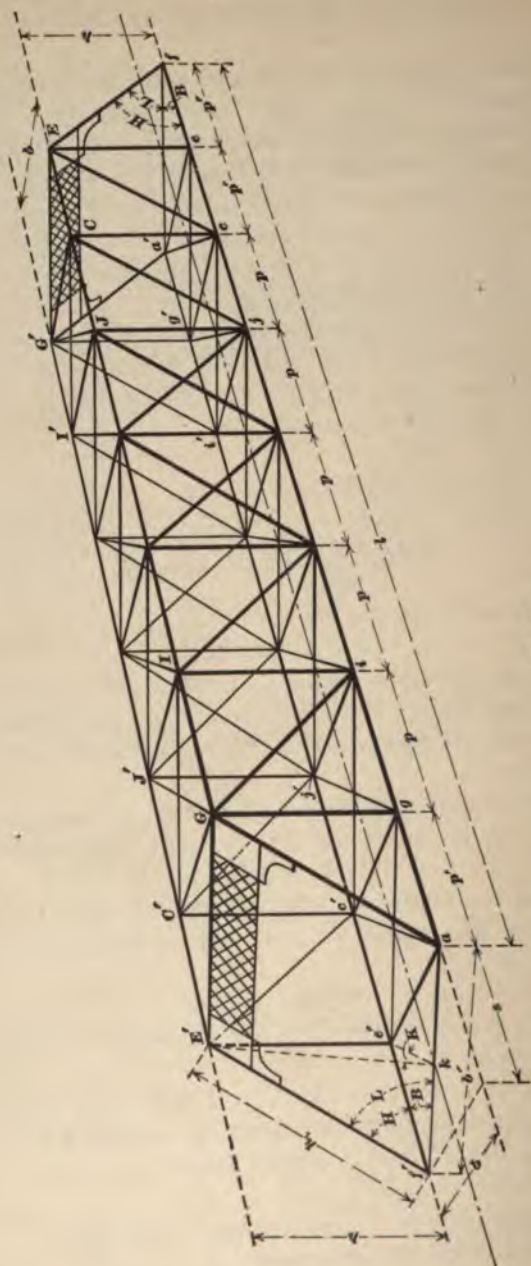


FIG. 19

The shear in $a N'$ is X_1 .

The moment at N' is $X_1 [(h_2 - d - d_1) \csc L]$.

To find the value of the angle L in terms of quantities that are known, it is well to consider Fig. 19, which is a perspective drawing of the bridge represented in Fig. 17. In Fig. 19, $f' E' G a$ and $f E G' a'$ are the planes of the portals. At the left end, the line $e' k$ is horizontal and perpendicular to $f' a$; then, the triangle $e' k f'$ is a right triangle. As $E' e'$ is vertical, the triangle $E' e' k$ is also a right triangle. If the line $e' k$ is revolved about the line $a f'$ and kept always perpendicular to it, it will generate a plane at right angles to $a f'$ that intersects the plane of the portal in $k E'$; the triangle $f' k E'$ is, therefore, a right triangle. Then,

$$\cos L = \frac{f' k}{E' f'}; f' k = e' k \cot B = b \cos B \cot B;$$

$$E' f' = h_1 = h \csc H;$$

whence $\cos L = \frac{b}{h} \cos B \cot B \sin H$

It is sometimes desired to find the angle $E' k e'$ between the plane of the portal and a horizontal plane. Let K represent the angle $E' k e'$; then,

$$\tan K = \frac{E' e'}{e' k}; E' e' = h; e' k = b \cos B;$$

whence $\tan K = \frac{h}{b} \sec B$

When K is known, L may also be found as follows:

$$\sin L = \frac{E' k}{E' f'}; E' k = h \csc K; E' f' = h_1 = h \csc H;$$

whence $\sin L = \sin H \csc K$

If the end posts are fixed at their lower joints, the stresses in the end post and portal may be found by assuming that the end posts are supported and hinged at points one-third to one-half the distance from their lower joints to the lowest point of the portal or brackets.

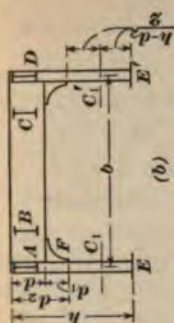
VIADUCTS

33. Definition.—A viaduct is a bridge, commonly of stone or steel, made up of several spans that rest on intermediate supports, and is used to carry elevated railways through city streets, or to carry a highway or railroad across a valley or stream, generally at a considerable distance above it. The stresses in each span may be analyzed according to the principles already discussed, and the supports are all that remain to be considered.

34. Elevated Railways.—In elevated railways, the main girders or trusses that carry the track are usually connected to cross-girders that rest on columns; the latter are supported on masonry piers at or about the level of the ground, and are firmly bolted to them, and so may be considered fixed in direction.

A typical elevation and section of such a structure built for two tracks is shown in Fig. 20. The cross-girders transmit the loads at *B* and *C* to the columns at *A* and *D*, and receive their maximum stresses when both tracks are loaded on the two adjacent spans. The columns transmit the vertical loads from the cross-girders and the outside-track girders to the piers, and also receive their maximum loads when the adjacent spans are fully loaded.

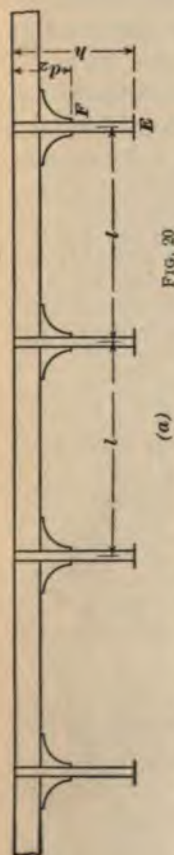
35. Column Bracing.—As the clearance between the columns below the cross-girders is usually required for street traffic, it is impossible to insert any diagonal sway bracing to resist the wind pressure and centrifugal force. Whenever possible, knee braces or curved brackets are inserted under the cross-girders. Then the two columns and cross-girder form a structure similar to the end posts and portal of a through bridge, and the direct, bending, and shearing stresses in the columns due to centrifugal force and to the



pressure of the wind on the cars and girders may be found in exactly the same way as for a portal. For this purpose, the formulas given in Fig. 10 may be used. In computing the wind pressure, it will be sufficient to consider the exposed area of one girder and one train of cars; the others will be sheltered. If the lower end of the column is firmly bolted to the masonry, the points of inflection C_1 and C_1' , Fig. 20 (b), may be assumed to be midway between the top of the pier and the bottom of curved bracket. Then, if P represents the total wind pressure over one span on cars and girders, the horizontal reactions at C_1' and C_1 are each equal to $\frac{P}{2}$, and the moment at F and F' is

$$\text{equal to } \frac{P}{2} \times \frac{h - d_2}{2}$$

FIG. 20



There is also to be resisted the longitudinal thrust exerted by a train of cars coming to rest when the brakes are set. This thrust is a maximum when the brakes are set so hard that the wheels stop turning and slide on the rails; it is usually taken equal to one-fifth the weight of the cars. It is transmitted to the piers by the columns, which thereby receive bending stresses. In the double-track structure shown in Fig. 20, the thrust for each track is transmitted by one row of columns; and in a single-track structure by both rows. If the longitudinal thrust per linear foot for one track is t , and the length of span l , the amount T that is transmitted by one column is tl , and, assuming that the columns are well bolted at the bottom, the bending moment at F is

$$T \times \frac{h - d_2}{2}$$

STEEL TRETTLES

36. General Description.—Fig. 21 shows the elevation

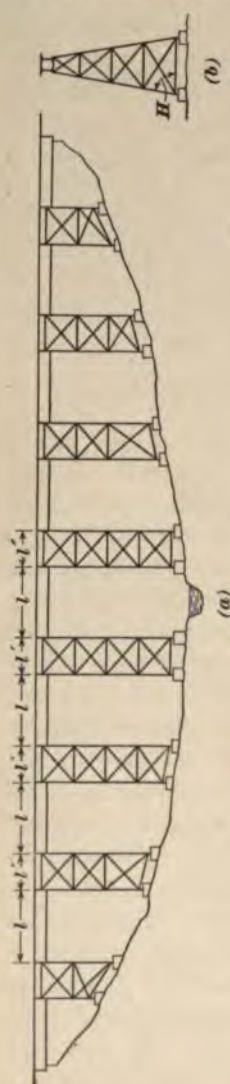


FIG. 21

and section (b) of a typical steel trestle for carrying a railroad or highway across a deep valley. The track or floor is carried on girders or trusses forming alternately long and short spans, l and l' , respectively. The adjacent ends of each two consecutive spans are supported by a pair of columns, the tops of which are about under the main girders or trusses. These columns are battered, or sloped outwards, toward the ground. The two columns supporting the corresponding ends of the two trusses or girders in each span are rigidly braced by horizontal struts and diagonal members in vertical planes at right angles to the track. The combination formed by the two columns and the bracing is called a **trestle bent**. The trestle bents that come under the two ends of each short span l' are connected to each other by horizontal struts and diagonal bracing in inclined planes parallel to the direction of the track. The two bents, together with the bracing, form a **tower**. The short spans, which extend from one side of the tower to the other, are called the **tower spans**, and are usually about 30 feet in length; the long spans, which extend from one tower to the next, are called the **main spans**, and are usually not less than about 50 feet in

length, the maximum depending to a great extent on the height of the columns.

The maximum vertical load on a bent occurs when the adjacent spans are fully loaded. If the length of the lower span is l' , that of the main span l , and the load per linear foot w , the vertical load W on a trestle bent is $w\left(\frac{l}{2} + \frac{l'}{2}\right)$, one-half of which is carried by each column. If the angle between the center line of the column and a horizontal line at right angles to the center line of the bridge is H , as represented in Fig. 22 (*b*), the compression in the column due to the vertical load is $\frac{W}{2} \csc H$, and the compression in the upper transverse strut AA' is $\frac{W}{2} \cot H$.

37. Transverse, or Sway, Bracing.—The diagonal bracing is usually arranged so that the slope is about 45° ; in a high tower, the bracing divides the height into a number of **stories**. The stresses in the bracing are usually found by the graphic method. For purposes of illustration, both the graphic and the analytic method will be explained here. Fig. 22 (*a*) represents a trestle bent with dimensions as shown; the wind pressure on the train for a length equal to $\frac{l}{2} + \frac{l'}{2}$ is P_1 , acting at a distance of h_1' above the top of the bent; and the wind pressure on the rails, ties, and girders, for the same length is P_2 , acting at a distance of h_2' above the top of the bent. The wind pressures transmitted to the joints A, B, C , etc. by the members of the bent are P_3, P_4, P_5 , etc. As the wind is blowing from the left, the diagonals $A'B, B'C, C'D$, etc. will be in action.

To find the stress in a diagonal, such as $A'B$, the bent may be considered cut by a plane, such as rs , Fig. 22 (*a*), that intersects the diagonal and the two columns, and the portion above the plane may be considered as a free body. Then, applying the equation $\sum M = 0$ to the external forces above the section rs , and taking the center of

moments at the intersection I of the two columns, the stress in $A'B$ is

$$\frac{P_1(i - h_1') + P_2(i - h_2') + P_3 i}{i_1}, \text{ tension}$$

In like manner, the compression in BB' is

$$\frac{P_1(i - h_1') + P_2(i - h_2') + P_3 i + P_4(i + h_1)}{i + h_1},$$

The stresses in other diagonals and transverse struts may be found in a similar way. For the stress in FF' , it will be well to consider that the whole horizontal load is carried to F' , in which case the stress in FF' will be equal to the moment of all the forces above the point F about I , including the load at F , divided by the lever arm $i + h$ of FF' .

The distances b , b_1 , etc., h , h_1 , h_2 , etc., and the angle H are determined by the style and general dimensions of the viaduct; from them, the distances i , i_1 , i_2 , etc. may be found as follows:

$$i = \frac{b}{2} \tan H$$

$$i_1 = IB \sin (H - H_1) = \frac{b_1}{2} \sec H \sin (H - H_1)$$

$$i_2 = \frac{b_2}{2} \sec H \sin (H - H_2), \text{ etc.}$$

38. Wind Stresses in Columns.—To find the wind stresses in the columns, the portion of the bent above such a section as rs may be considered. The stress in either column may be found by taking moments about the intersection of the diagonal with the other column. It will be convenient to resolve the stress into its X and Y components at the joint opposite the center of moments, and to multiply the Y component by $\csc H$. Thus, with the center of moments at A' , the stress in AB is

$$\frac{P_1 h_1' + P_2 h_2'}{b} \times \csc H, \text{ tension}$$

With the center of moments at B , the stress in $A'B'$ is

$$\frac{P_1(h_1' + h_1) + P_2(h_2' + h_2) + P_3 h_1}{b_1} \times \csc H, \text{ compression}$$

With the center of moments at F , the stress in $E'F'$ is compression equal to

$$\{[P_1(h_1' + h) + P_2(h_2' + h) + P_3h + P_4(h - h_1) + P_5(h_3 + h_4 + h_5) + P_6(h_6 + h_7) + P_7h_8] \div b_1\} \times \csc H$$

The stress in any other portion of the columns may be found in like manner. The maximum compression will occur at the lowest section on the leeward side, in this case $E'F'$, and will be equal to the compression due to the vertical load plus that due to wind pressure; this is also the maximum vertical load on a pier.

For the minimum compression in the columns, there are two cases to be considered.

1. *When the adjacent spans are fully loaded.* In this case, the compression due to the live and dead loads is decreased by the tension in the windward column due to a wind pressure of 30 pounds per square foot against the exposed surface of the cars and structure.

2. *When the adjacent spans are unloaded.* In this case, the compression due to dead load alone is decreased by the tension in the windward column, due to a wind pressure of 50 pounds per square foot against the exposed surface of the structure alone. The minimum compression occurs in the lowest section of the windward column, in this case EF , but this is not the minimum vertical load on the pier; the latter is further decreased by the vertical component of the tension in $E'F'$, so that the minimum load on the windward pier may be negative; that is, there may be a tendency to uplifting at F , even though the minimum stress in EF is compression.

39. Anchorage.—To find the reaction on the windward pier, without considering the stresses in the members, the point F' may be taken as the center of moments for all the external forces acting on the bent. Then, denoting by W the total load on a bent, and assuming that the reaction at F acts vertically upwards, its value is

$$W \times \frac{b_1}{2} - \text{moment of all wind pressure about } F'$$

b_1

If this comes out positive for both cases outlined in the preceding example, no anchorage is required; if it comes out negative for either case, there must be a weight of masonry in the pier equal at least to the negative reaction, although for safety it is usually made not less than twice that amount.

40. Graphic Method.—The stresses may be found by means of the stress diagram much more quickly than by the analytic method. Those due to the vertical load may be found by drawing the triangle of forces, Fig. 23 (*d*), for a load of $\frac{W}{2}$, and the stresses in AA' and AB at point A .

In finding the wind stresses, it will be well to consider first the pressures P_1 and P_2 that act above the top of the bent. The lateral thrust due to P_1 is transmitted to the upper flanges of the girders at a and a' , Fig. 22, by means of the rails, ties, and upper lateral bracing; that due to P_2 is transmitted to a and A by means of the web of the girder; the lateral thrust at a and a' is transmitted to the supports at A and A' by means of the sway frame $aa'A'A$, the diagonal aA' being out of action, and there being tension in $a'A$. In the analysis of the sway frame, in Art. 20, it was explained that it is customary to consider the structure on the point of overturning about the leeward support, in which case the whole lateral thrust is exerted at A' , causing a compression in the lower member of the sway frame equal to $P_1 + P_2$.

Now, if the same assumption is made in the present case, there will be very little stress in AA' , the upper transverse strut of the tower, and to furnish a basis for the design of this member it is assumed that all the lateral thrust due to P_1 and P_2 is transmitted to the bent at A ;^{*} then, the reaction at A' will be vertical. The problem then arises to resolve P_1 and P_2 into two components, one of which

^{*}In reality, it matters little whether all the lateral thrust is assumed to be transmitted to A or to A' , as all the transverse struts of the bent, including the top, are usually made the same size.

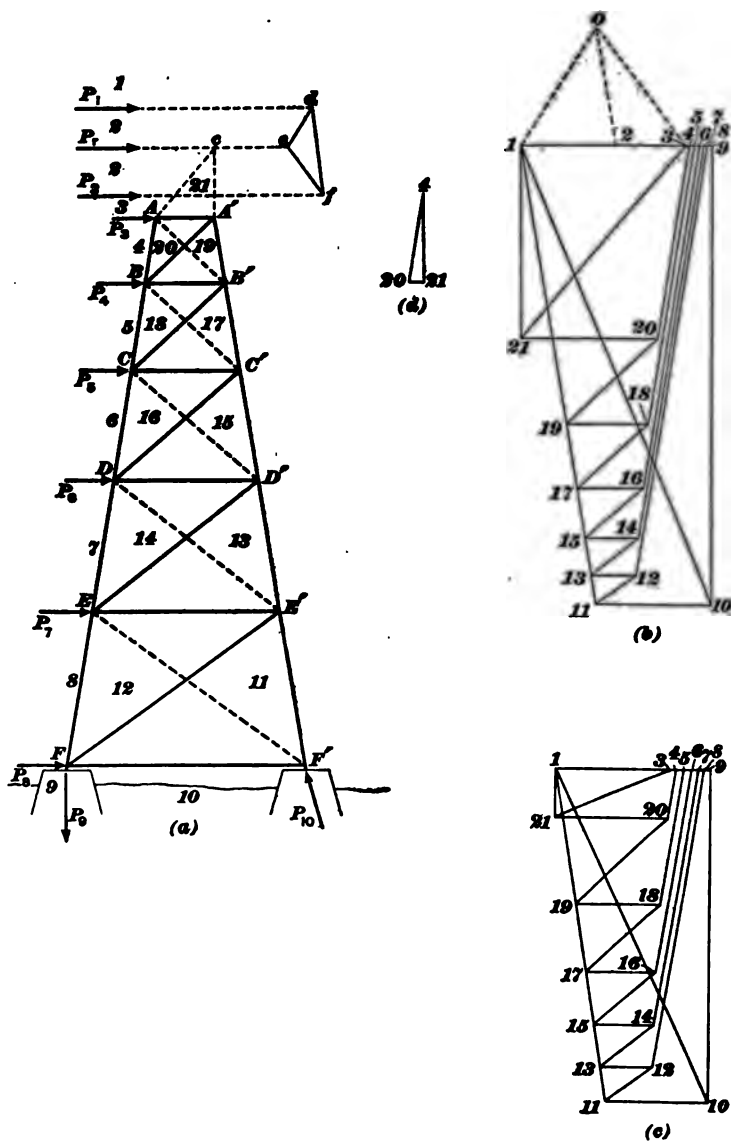


FIG. 23

is vertical, passing through A' , and the other inclined, passing through A . For this purpose, P_1 and P_2 are platted to scale, as shown by 1-2 and 2-3, respectively, in Fig. 23 (*b*), and the position of their resultant $P_r = P_1 + P_2$ is found by means of the force diagram 0-1-2-3, Fig. 23 (*b*), and the funicular $defd$, Fig. 23 (*a*). The line of action of P_r intersects at c the line of action of the vertical reaction at A' ; therefore, cA is the line of action of the reaction at A . The values of the reaction may be found by drawing the triangle of forces for the point c ; they are 1-21 at A' , and 21-3 at A , Fig. 23 (*b*). Then, the external forces acting on the tower are: 1-21, acting vertically downwards at A' ; 21-3, acting upwards in the direction Ac at A ; P_3, P_4, P_5 , etc., acting horizontally at A, B, C , etc.; the reaction at F , which may be assumed vertical, as explained in Art. 39; and the reaction at F' , which will be inclined. The diagonals $A'B, B'C, C'D$, etc. will be in tension; the other diagonals will be out of action. The construction of the stress diagram presents no difficulty; it is best to start with the joint A , then take A' , and then B, B', C, C', D, D' , etc. in the order given. In Fig. 23, (*a*) is the elevation or space diagram of the trestle bent; (*b*), the stress diagram for wind stresses when the structure is fully loaded (30 pounds per square foot against structure and cars); (*c*), the stress diagram for wind stresses when the structure is unloaded (50 pounds per square foot against the structure); and (*d*), the force polygon for the vertical load at the joint A . The stresses scaled from (*b*) must be combined with those due to full dead and live loads, and those scaled from *c* must be combined with those due to dead load alone; the maximum must be used in each case.

41. Centrifugal Force.—The stresses in the members of the bents due to centrifugal force when the viaduct is on a curve may be found in practically the same way as the wind stresses.

42. Longitudinal Thrust.—The bracing referred to in Art. 36, which connects the bents, is the longitudinal

bracing, and is designed to resist the longitudinal thrust that is exerted on the structure by a train of cars. The total thrust T that acts on one tower is equal to $t \left(\frac{l}{2} + l' + \frac{l}{2} \right)$, or $t(l + l')$. It is transmitted to the viaduct at the level of the rails, and may be assumed to be concentrated at the top of one bent, or divided between the two; for all practical purposes, it may be assumed to be applied at one bent, as at G , Fig. 24. The amount to be resisted by the bracing on

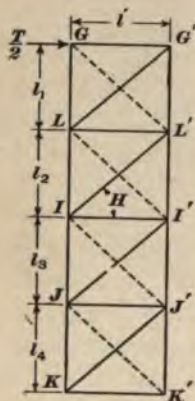


FIG. 24

each side of the track is $\frac{T}{2}$; the stress in

each diagonal is equal to $\frac{T}{2} \times \sec H$, tension, and in each horizontal strut it is $\frac{T}{2}$, compression. The stresses in the

columns may be found by the method of moments. For example, the stress in

$$LI \text{ is } \frac{\frac{T}{2} \times l_1}{l'}, \text{ tension; and the stress in}$$

$$L'I' \text{ is } \frac{\frac{T}{2}(l_1 + l_2)}{l'}, \text{ compression.}$$

The distances l_1, l_2, l_3 , etc. are not vertical heights, but actual distances measured along the columns.

EXAMPLE.—In Fig. 22, let $h_1' = 15$ feet; $h_2' = 4$ feet; $h_1, h_2, h_3, h_4, h_5 = 9, 12, 15, 18, 21$ feet, respectively (then $h = 75$ feet); $b = 8$ feet; slope of columns, 2 inches horizontal to 1 foot vertical; $P_1 = 15,000$ pounds; $P_2 = 12,000$ pounds; and let the wind pressure on the bent be 100 pounds per linear foot, vertical. Determine: (a) the vertical reaction at F and the vertical and horizontal components of the reaction at F' ; (b) the wind stress in the member $E'F'$; (c) the wind stress in the member EF ; (d) the wind stress in the member $C'D$; and (e) the wind stress in the member CC' .

SOLUTION.—(a) The vertical reaction at F is found by taking moments of all the wind forces about F' . The moments, in foot-pounds, are:

$P_1(h + h_1')$	$= 15,000 \times (75 + 15)$	$= 1\ 3\ 5\ 0\ 0\ 0$
$P_2(h + h_2')$	$= 12,000 \times (75 + 4)$	$= 9\ 4\ 8\ 0\ 0\ 0$
P_3h	$= \frac{9 \times 100}{2} \times 75$	$= 3\ 3\ 7\ 5\ 0$
$P_4(h - h_1)$	$= \frac{21 \times 100}{2} \times (75 - 9)$	$= 6\ 9\ 3\ 0\ 0$
$P_5(h - h_1 - h_2)$	$= \frac{27 \times 100}{2} \times (75 - 9 - 12)$	$= 7\ 2\ 9\ 0\ 0$
$P_6(h_3 + h_4)$	$= \frac{33 \times 100}{2} \times (21 + 18)$	$= 6\ 4\ 3\ 5\ 0$
P_7h_5	$= \frac{39 \times 100}{2} \times 21$	$= 4\ 0\ 9\ 5\ 0$
$P_8 0$	$= \frac{21 \times 100}{2} \times 0$	$= 0$
		$2\ 5\ 7\ 9\ 2\ 5\ 0$

The total moment about F' is 2,579,250 ft.-lb. Each column slopes outwards $\frac{3}{12} \times 75 = 12.5$ ft.; then,

$$b_s = 12.5 + 8 + 12.5 = 33 \text{ ft.}$$

The vertical reaction at F is, therefore,

$$2,579,250 \div 33 = 78,200 \text{ lb., acting downwards. Ans.}$$

Since the only vertical forces acting on the trestle bent for the wind loading are the vertical components of the reactions at F and F' , they must be equal. The vertical component of the reaction at F' is, therefore, 78,200 lb. Ans.

In Art. 37, it is stated that the entire horizontal load may be assumed to be carried to F' . Then, the horizontal component of the reaction at F' is

$$15,000 + 12,000 + (75 \times 100) = 34,500 \text{ lb. Ans.}$$

(b) The stress in the member $E'F'$ can be found by taking moments about the intersection of EF and $E'F$, that is, about point F . This is the same moment found in (a), 2,579,250 ft.-lb.

$$\csc H = \frac{\sqrt{12^2 + 2^2}}{12} = 1.0138$$

Then, the stress in $E'F'$ is

$$\frac{2,579,250}{33} \times 1.0138 = +79,300 \text{ lb. Ans.}$$

(c) The wind stress in the member EF can be found by taking moments about the intersection of $E'F'$ and $E'F$, that is, about point E' . The moments, in foot-pounds, are:

$P_1(h_1 + h_2 + h_3 + h_4 + h_1')$	$= 15,000 \times 69$	$= 1\ 0\ 3\ 5\ 0\ 0\ 0$
$P_2(h_1 + h_2 + h_3 + h_4 + h_2')$	$= 12,000 \times 58$	$= 6\ 9\ 6\ 0\ 0\ 0$
$P_3(h_1 + h_2 + h_3 + h_4)$	$= 450 \times 54$	$= 2\ 4\ 3\ 0\ 0$
$P_4(h_3 + h_4)$	$= 1,050 \times 45$	$= 4\ 7\ 2\ 5\ 0$
$P_5(h_3 + h_4)$	$= 1,350 \times 33$	$= 4\ 4\ 5\ 5\ 0$
P_6h_5	$= 1,650 \times 18$	$= 2\ 9\ 7\ 0\ 0$
		$1\ 8\ 7\ 6\ 8\ 0\ 0$

The distance b_1 is

$$E E' = \left(2 \times \frac{54 \times 2}{12} \right) + 8 = 26 \text{ ft.}$$

Then, the stress in EF is

$$\frac{1,876,800}{26} \times 1.0138 = -73,200 \text{ lb.}$$

(d) The stress in $C'D$ is equal to the moment of P_1, P_2, P_3, P_4 , and P_5 about I divided by the perpendicular distance i_2 from I to DC' produced.

$$i = \frac{b}{2} \times \tan H = \frac{8}{2} \times 6 = 24 \text{ ft.}$$

FOOT-POUNDS

$P_1(i - h_1')$	$= 15,000 \times 9 = 135000$
$P_2(i - h_2')$	$= 12,000 \times 20 = 240000$
$P_3 i$	$= 450 \times 24 = 10800$
$P_4(i + h_1)$	$= 1,050 \times 33 = 34650$
$P_5(i + h_1 + h_2)$	$= 1,350 \times 45 = 60750$
	<hr/> 481200

According to Art. 37,

$$i_2 = \frac{b_2}{2} \times \sec H \sin (H - H_2)$$

In this equation,

$$b_2 = \left(2 \times \frac{36 \times 2}{12} \right) + 8 = 20 \text{ ft.}$$

$$\sec H = \frac{\sqrt{12^2 + 2^2}}{2} = 6.0828;$$

$$H = 80^\circ 32';$$

$$\tan H_2 = \frac{h_2}{\frac{b_2}{2} + \frac{b_1}{2}} = \frac{2 h_2}{b_2 + b_1} = \frac{2 \times 15}{20 + 15} = .85714$$

so that $H_2 = 40^\circ 36'$. Then,

$$i_2 = \frac{20}{2} \times 6.0828 \times \sin (80^\circ 32' - 40^\circ 36') = 60.828 \times .64190 = 39.05 \text{ ft.}$$

The stress in $C'D$ is, therefore,

$$481,200 \div 39.05 = -12,300 \text{ lb. Ans.}$$

(e) The stress in CC' is equal to the moment of P_1, P_2, P_3, P_4 , and P_5 about I , divided by the perpendicular distance from I to CC' , which is 45 ft. The moment was found, in (d), to be 481,200 ft.-lb. Then, the stress in CC' is

$$481,200 \div 45 = +10,700 \text{ lb. Ans.}$$

EXAMPLES FOR PRACTICE

1. Suppose that the wind pressure P_1 on the trestle bent shown in Fig. 22, when there is no live load on the trestle, is 20,000 pounds; the wind pressure on the bent is 150 pounds per foot vertical; and the

dimensions are as given in the example in Art. 42. Find the vertical reaction at F . Ans. 60,700 lb.

2. For the conditions stated in example 1, what is the horizontal component of the reaction at F' ? Ans. 31,250 lb.

3. For the same conditions as in example 1, what is the stress in the member $D'E'$? Ans. + 53,800 lb.

4. For the same conditions as in example 1, what is the stress in the member $C'D$? Ans. - 14,300 lb.



BRIDGE MEMBERS AND DETAILS

(PART 1)

MATERIALS

1. The materials most used in the construction of bridges are wood, iron, and steel. In this Section, the materials and the forms and shapes specially adapted to bridge work will be considered. The specifications governing the quality of materials, and the details of construction of bridges, will be treated in subsequent Sections.

WOOD

2. In the first bridge trusses that were built, wood was used almost wholly for compression members; to a great extent for tension members, such as lower chords; and for floor systems. As the loads at that time were light, and timber was plentiful in almost all localities, while iron and steel were expensive and of uncertain quality, wood was considered the most desirable material. With the advance of civilization, loads increased in weight, the clearing of large areas made timber scarce and expensive, and new methods of manufacture rendered iron and steel more reliable and less expensive. On this account, when the timber bridges first built deteriorated to such an extent that it became necessary to renew them, many were replaced by structures entirely of iron or steel, with the exception, in many cases, of the floor system, for which wood still remained the most satisfactory material.

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3. At the present time, timber is used to some extent for the compression members of trusses in which the tension members are steel, in localities where timber is still plentiful and inexpensive. Such bridges are known as **combination bridges**, and the details are arranged in such a way that the wooden compression members can be readily removed and replaced when renewal is necessary. In by far the greater number of bridges at the present time, however, the floor planking, and, in some cases the stringers and floorbeams, are the only parts constructed of wood.

IRON

CAST IRON

4. **Properties.**—Cast iron is a product obtained by melting iron ore, and consists of a mixture of iron and carbon, together with small percentages of other impurities, such as sulphur, phosphorus, manganese, and silicon. The relatively large amount of carbon, varying from 2 to 6 per cent., distinguishes it from other grades of iron. Cast iron is comparatively cheap, has a high compressive strength, can be given any desired shape very readily by casting, and does not rust to any appreciable extent. These properties recommended it for use in bridge trusses, when it was first deemed advisable to use some material other than wood. On the other hand, cast iron has a comparatively low and uncertain tensile strength; its high percentage of carbon renders it very brittle, causing it to break abruptly without any warning; and the method of manufacture is such that there may exist in the material hidden flaws that greatly impair its strength. On account of its brittleness, cast iron cannot be riveted, as the pressure of a riveting machine or the blows from a riveting hammer are almost sure to break the iron.

5. **Use of Cast Iron.**—When cast iron was first used in bridge construction, engineers did not attach sufficient importance to its unreliable qualities. Several disastrous

bridge failures, however, soon led to the abandonment of this material for important members. For a number of years, it has been used only for unimportant parts, such as heavy bedplates or masonry plates under the ends of trusses, to distribute the pressure over the required area of the masonry. At the present time, it is being entirely superseded in bridge work by steel, even for unimportant details. It is a good rule *never to use cast iron for any part of a bridge the failure of which may cause damage to life or property.*

WROUGHT IRON

6. Properties.—Wrought Iron is the product obtained by burning the carbon, and, to a certain extent, the other impurities from cast iron. The result is a porous mass of pure iron with some impurities; the smaller the quantity of these impurities, the better is the quality of the iron. The first product of the burning is hammered and pressed between rolls into a compact and homogeneous mass, to eliminate the pores and weld the iron firmly into a solid mass. The amount and method of hammering and rolling determine to a great extent the quality of the iron.

Wrought iron has a lower compressive strength than cast iron, and is more expensive; on the other hand, it possesses so many advantages over cast iron that it is far preferable in bridge work. By successive heating and hammering, it can be given almost any desired form; two pieces of it can readily be welded or riveted with very little injury to the iron; it is very tough and ductile, and gives ample warning before failure.

7. Use of Wrought Iron.—In the earlier bridge trusses, wrought iron was used for the tension web members, and later on replaced timber and cast iron in all the members, being used for several years with fairly good results. As the loads increased, it was necessary to build the bridges stronger; this could have been done by using heavier members of wrought iron, but, on account of the high cost of such members, it became very desirable to replace this

material by another having greater strength, and not inferior in other properties. Steel was found to possess the needed requirements, and has superseded wrought iron in nearly all forms of structural work.

8. The earlier grades of wrought iron were of comparatively low quality; the material lacked homogeneity, and possessed all the impurities of the cast iron from which it was made. With modern appliances for the manufacture of iron and steel, a much better quality can be obtained, and it is thought by some engineers that steel is used at present in some places where the conditions are such that wrought iron would give much better results. This is particularly true of highway bridges over railway tracks, where, on account of the sulphurous fumes from the smokestacks of locomotives, steel corrodes very rapidly. There are several instances where second-hand wrought-iron trusses supporting new steel floor systems over railway tracks were in excellent condition, so far as corrosion was concerned, after the steel floorbeams and stringers had corroded to such an extent that it was necessary to replace them. As a general rule, however, steel is far preferable to wrought iron for bridge work; wrought iron is used at present simply for members that require to be welded, and for details that cannot easily be manufactured of steel. The grades of steel used for bridge work do not weld well.

STEEL

9. **Method of Manufacture.**—Steel is manufactured in three ways; namely, by adding carbon to wrought iron, by removing carbon from cast iron, or by mixing cast and wrought iron in suitable proportions. The first process, which is called the **crucible process**, is the most expensive. Crucible steel is never used in bridge work.

The second process is known as the **Bessemer process**, and gives a grade of steel that is used to a great extent in structural work. Owing to the method of manufacture, Bessemer steel is liable to have hard and brittle spots, and

is not homogeneous. For this reason, it is not well adapted for use in members subject to shocks and blows, such as bridge members, although it is sometimes used.

The third process is known as the **open-hearth process**, and gives a grade of steel similar to Bessemer steel, but more homogeneous. Open-hearth steel is especially adapted to bridge work, and is largely used for that purpose; it is somewhat more expensive than Bessemer steel, but the additional expense is thought to be justified by the greater homogeneity obtained in the metal.

10. Red Shortness and Cold Shortness.—The strength and ductility of steel depend to a great extent on the amount of impurities, those usually present being silicon, manganese, carbon, sulphur, and phosphorus. The first two are sufficiently removed in modern methods of manufacture, and will not be further discussed. The amount of carbon can be regulated at will, according to the tenacity and ductility required, and varies from .1 to 1 per cent. of the product. In general, the lower the percentage of carbon, the softer is the steel; the higher the percentage of carbon, the harder is the steel.

The presence of sulphur makes the steel **red short**; that is, brittle and easily cracked when hot. This is a very undesirable property, as steel is heated and worked several times during the process of manufacture, and if much sulphur is present, the material is broken and wasted.

The presence of phosphorus makes steel **cold short**; that is, brittle and easily broken when cold. This, too, is a very undesirable property in steel that is to be used in bridge work, where the members are subject to shocks, and where the failure of a member may cause disastrous results.

For the reasons just stated, it would be desirable to remove sulphur and phosphorus entirely from steel; but, as the necessary process is too expensive, it is customary to require that the amounts of them shall not exceed a certain percentage of the product. The amount of sulphur allowed in steel for bridge work is usually not more than .04 per cent.; that of

phosphorus varies from .04 to .08 per cent., according to the method of removing the phosphorus.

11. Acid Process and Basic Process.—There are two methods of manufacture by both the Bessemer and the open-hearth process; these methods have been given the names **acid** and **basic**, the difference consisting in the lining of the receptacle in which the steel is prepared and the materials that are added during the process of preparation.

In the acid process, the receptacle is lined with silicious sandstone, and pig iron with a small percentage of phosphorus is used; none of the latter is removed by this method, the amount in the finished steel being the same as in the pig iron put in the receptacle.

In the basic process, the receptacle is lined with dolomite (magnesium limestone), and pig iron with a large amount of phosphorus may be used, as substances are added to the pig iron that remove most of the phosphorus.

It has been found in practice that, if steel is manufactured by the acid process, the amount of phosphorus should not exceed .08 per cent.; if made by the basic method, the phosphorus should not exceed .04 per cent.

12. Melts.—The quantity of steel that is manufactured in one operation of the melting furnace is spoken of as a **melt**. Each melt is numbered so that the record can be easily found by referring to the melt number. The chemical properties of each melt are placed on record for future reference as soon as the melt is finished.

13. When the proper chemical composition of the steel has been obtained, the molten metal in the receptacle (that is, the melt) is poured into molds, forming large castings called **ingots**. These ingots are sometimes rolled a few times to form what are known as **billets**. All the ingots or billets that come from one melt are stamped with the melt number. The billets are subsequently reheated and passed successively between rolls, each rolling decreasing the cross-section and increasing the length of the billet, until the desired form is reached. In this way, a great variety of shapes—such as

rods, plates, angles, channels, I beams, etc.—are obtained. The melt number on a billet should be stamped on each piece rolled from that billet.

The tenacity and ductility of the finished material are ascertained by certain mechanical tests prescribed by the bridge engineer in charge of the work and made under the supervision of inspectors appointed by him. The usual tests that are prescribed will be given in *Bridge Specifications*.

STRUCTURAL SHAPES OR SECTIONS

INTRODUCTION

14. Properties of Sections.—In *Strength of Materials*, the moment of inertia, radius of gyration, and section modulus have been defined, and the methods of computing them explained. In actual practice, it is seldom necessary to compute these properties for the simple rolled shapes, as they are computed by the steel manufacturers, who publish books containing tables in which the results of such computations are given. Several of these tables, and others that are convenient for bridge work, are given in *Bridge Tables*. All references in this Section are to those tables, unless otherwise stated.

15. Weight of Steel.—In *Bridge Tables*, the weight of a cubic foot of steel has been taken as 489.6 pounds. As the weight of 1 cubic foot of material is equal to that of 144 pieces 1 inch square and 12 inches long, the weight of each of these pieces of steel is $489.6 \div 144 = 3.4$ pounds. That is, *the weight of a piece of steel 1 inch square and 1 foot long is 3.4 pounds, and, therefore, the weight per linear foot of any piece of steel of uniform cross-section may be found by multiplying the area of the cross-section, in square inches, by 3.4.*

For example, the area of cross-section of a steel plate 12 inches wide and $\frac{1}{2}$ inch thick is 6 square inches; the weight of the plate, per foot, is, therefore, $6 \times 3.4 = 20.4$ pounds.

SIMPLE ROLLED SHAPES

SQUARE AND ROUND RODS

16. Areas and Weights.—Table I gives the areas of cross-section and weights per linear foot for round rods from $\frac{1}{8}$ inch to 6 inches in diameter; and Table II gives corresponding values for square rods from $\frac{1}{8}$ inch to 6 inches on a side. For example, it can be seen at a glance that the area of cross-section of a round rod $2\frac{1}{8}$ inches in diameter is 6.49 square inches, and that the weight per foot is 22.07 pounds. These rods are frequently used for tension members, such as lateral rods or counters, but are seldom used for the main members of trusses. The rods are sometimes connected at the ends by means of screw threads on which are turned nuts or other devices for transmitting to the rods the forces they are to resist.

17. Upset Screw Ends.—If screw threads are cut on the ends of a round rod, the area of cross-section at the root of the thread is considerably less than that of the body of the rod, and the screw ends are not so strong as the remainder of the rod. On this account, it is customary to enlarge the ends before the threads are cut; this is done by a process called **upsetting**. The rod, as originally made, is the same diameter throughout its entire length; in the process of upsetting, the ends are heated, and the diameter at the ends is enlarged to such an extent that, after the threads are cut on the enlarged ends, the area of cross-section at the root of the thread will be greater than that of the body of the bar; the greater area is necessary because the process of upsetting somewhat weakens the steel at the ends. When it is desired to form screw ends on square rods, the ends are upset to cylindrical forms. The rod is considerably shortened in upsetting, and so it is necessary to allow for the shortening effect.

Table III gives the dimensions of standard upset screw ends for round rods from $\frac{3}{4}$ inch to 3 inches in diameter, and

for square rods from $\frac{3}{4}$ inch to 3 inches on a side, together with the additional lengths of the original rods necessary to form a screw end. These dimensions give an area of cross-section at the root of the thread from 20 to 50 per cent. greater than that of the body of the bar, and have been found to be satisfactory in actual practice. It is seldom necessary in bridge work to upset a round rod greater than 3 inches in diameter, or a square rod greater than 3 inches on a side.

18. Hexagon Nuts.—Table IV gives the standard dimensions and weights of hexagon nuts, which are commonly used with rods with screw ends.

EXAMPLE.—A round rod $2\frac{3}{8}$ inches in diameter is required to be 10 feet long and to have two upset ends. (a) What is the diameter of the screw end? (b) What is the length of the screw end? (c) How long must the rod be before upsetting?

SOLUTION.—(a) Table III gives the diameter D of the screw end for a round rod $2\frac{3}{8}$ in. in diameter as 3 in. Ans.

(b) Table III gives the length L of the screw end for the same rod as 6 in. Ans.

(c) Table III gives the additional length U of rod necessary to form one upset end as $4\frac{3}{8}$ in. As there are two upset ends, the original length of the rod must be

$$10 \text{ ft.} + (2 \times 4\frac{3}{8} \text{ in.}) = 10 \text{ ft. } 8\frac{3}{4} \text{ in.} \quad \text{Ans.}$$

FLAT PLATES

19. Areas and Weights.—Table VI gives the areas of cross-section, and Table VII gives the weights per linear foot, of steel plates from $\frac{1}{8}$ to 1 inch in thickness and from $\frac{1}{4}$ inch to 100 inches in width. The tables contain the usual sizes of plates; the areas and weights for other sizes can be found from these very readily by interpolation. Plates are used for webs and flanges of plate girders, and, in connection with other shapes, for tension and compression members in trusses.

20. Extreme Lengths of Plates.—Table V gives, approximately, the greatest lengths of various sizes of steel

plates that can be furnished by the rolling mills; these lengths should not be exceeded in designing bridges. In case plates are desired longer than the lengths given in the table, they can be formed by splicing together several pieces of shorter lengths, as explained in subsequent articles. In consulting Table V, if the width and thickness desired are not given in the table, it is necessary to use the length given for the next greater width and thickness.

21. Moments of Inertia of Rectangular Sections.

Table VIII gives the values of the moments of inertia, about axes at right angles to the width, of rectangular sections from $\frac{1}{4}$ to 1 inch in thickness and from 2 to 60 inches in width. The value for any other thickness can be found very readily from those given in the table. As will be seen later, this table is extremely useful in calculating the properties of compound or built-up shapes.

EXAMPLE.—Assuming a steel plate 56 inches in width and $\frac{1}{8}$ inch in thickness: (a) what is the area of its cross-section? (b) what is its weight per linear foot? (c) what is the greatest length the rolling mills can furnish? (d) what is the value of its moment of inertia about an axis perpendicular to the width?

SOLUTION.—(a) As 56 is not given in the list of widths in Table VI, it will be well to consider the given plate equivalent to two plates, one of which is 50 in. and the other 6 in. in width. From Table VI, the area of cross-section of a plate 50 in. wide and $\frac{1}{8}$ in. thick is found to be 34.375 sq. in.; and the area of cross-section of a plate 6 in. wide and $\frac{1}{8}$ in. thick is found to be 4.125 sq. in. The sum of these, or 38.5 sq. in., is the area of cross-section of the given plate. Ans.

(b) Proceeding as in (a), we find, from Table VII, the weight per linear foot of a plate 6 in. in width and $\frac{1}{8}$ in. in thickness to be 14.03 lb., and that of a plate 50 in. in width and $\frac{1}{8}$ in. in thickness to be 116.9 lb. The sum of these, or 130.9 lb., is the weight per linear foot of the given plate. Ans.

(c) As, in Table V, 56 is not given in the list of widths, nor $\frac{1}{8}$ in the list of thicknesses, the next greater width, in this case 60 in., and the next greater thickness, in this case $\frac{3}{8}$ in., are looked for. The length corresponding to this width and thickness is 30 ft. Then, 30 ft. is approximately the longest $56'' \times \frac{1}{8}''$ plate that can be had.

(d) Consulting Table VIII, the value 10,061 for the moment of inertia is found opposite a width of 56 in. and below a thickness of $\frac{1}{8}$ in. Ans.

ANGLES

22. Properties.—Table IX gives the dimensions, areas of cross-section, weights per linear foot, and other useful properties of standard angles having equal legs; and Table X gives the same properties of angles having unequal legs. In bridge work, angles are employed more than any other single shape, with the exception of plates, and are used for flanges of plate girders, members of lateral trusses, and, in connection with other sections, for many built-up tension and compression truss members. The smaller sizes of angles, especially those having legs less than $2\frac{1}{2}$ inches in width, are never used except for unimportant details.

An angle is usually referred to by a product of three numbers, the first two of which express the widths of the legs, and the other the thickness. Thus, an $8'' \times 8'' \times \frac{3}{4}''$ angle is an angle each of whose legs is 8 inches wide and whose thickness is $\frac{3}{4}$ inch; a $6'' \times 4'' \times \frac{1}{2}''$ angle is an angle one of whose legs is 6 inches wide, the other 4 inches, and whose thickness is $\frac{1}{2}$ inch. In the case of unequal-leg angles, the width of the wider leg is usually written or named first, as in the example just given.

The symbol **L** is commonly used for *angle*, and **LS** for *angles*. Thus, 2 **LS** $6'' \times 4'' \times \frac{1}{2}''$ means two $6'' \times 4'' \times \frac{1}{2}''$ angles.

23. Actual Size and Nominal Size of Legs.—Angles are made by heating and rolling steel billets. The finishing process consists in passing the steel through special sets of rolls that give it the required angular form. Each one of these sets of rolls is used for angles of various thicknesses and of approximately the same width. In the construction of tables, these widths are considered to be all equal, and to have a common value equal to the actual width of the thinnest angle for which that set of rolls is employed. This width, for angles other than the thinnest, is called their **nominal width**, to distinguish it from their actual width. For the purposes of selecting angles in the general design

of members, the nominal width, which is smaller than the actual width, is used; the error is small, and on the side of safety. For detailing, making connections, etc., it is necessary to take into account the difference between actual and nominal widths.

24. Table XI gives the thicknesses of angles for which the actual width is equal to the nominal. Opposite a width of 5 in. \times 3 in., for example, are found the thicknesses $\frac{5}{16}$ and $\frac{1}{2}$. This means that there are two sets of rolls for rolling 5" \times 3" angles: the thinnest angle that is rolled with one of these sets is $\frac{5}{16}$ inch thick; the actual widths of the legs of those angles are 5 and 3 inches. The same set of rolls is used for angles whose thicknesses lie between $\frac{5}{16}$ inch and $\frac{1}{2}$ inch; but, although these angles are called 5" \times 3" angles, they are a little wider than indicated by those figures. Likewise, the smallest thickness rolled with the other set of rolls is $\frac{1}{2}$ inch,

for which the actual widths are 5 and 3 inches; for greater thicknesses, the actual widths are slightly greater.

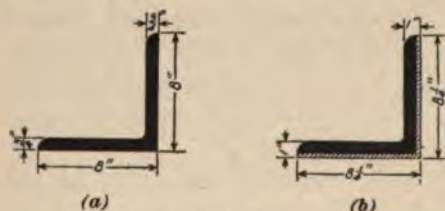


FIG. 1

Fig. 1 (a) shows a cross-section of an 8" \times 8" \times $\frac{3}{4}$ " angle.

According to Table XI, these are actual dimensions, since the angle has the minimum thickness for which the rolls are used. For an 8" \times 8" \times 1" angle, the same rolls are used, and are set $\frac{1}{4}$ inch farther apart to allow for the increase in thickness. The result is shown in Fig. 1 (b), the increase being shown by section lines. The actual size of this angle is, then, 8 $\frac{1}{4}$ in. \times 8 $\frac{1}{4}$ in. \times 1 in., although it is called an 8" \times 8" \times 1" angle. In general, the actual width of an angle can be obtained from the nominal width by the following

Rule.—Add to the nominal width the difference between the given thickness and the next smaller thickness for that width given in Table XI.

For example, the actual size of a $6'' \times 4'' \times \frac{3}{4}''$ angle is $6\frac{3}{16}$ in. $\times 4\frac{3}{16}$ in. $\times \frac{3}{4}$ in., since the next smaller thickness given in Table XI is $\frac{9}{16}$ inch, and the difference between $\frac{3}{4}$ and $\frac{9}{16}$ is $\frac{3}{16}$ inch.

25. Location of Gauge Lines.—Angles are connected to each other and to other shapes by rivets, the centers of which are located on lines, called **gauge lines**, parallel to the edges of the angles. In some cases, the rivets are located on one line, in others, on two, and sometimes, in 8-inch legs, on three lines. Table XII gives the standard distances from the gauge lines to the edges of the angles, together with the diameters of the largest rivets that can conveniently be driven into the leg.

CHANNELS

26. Channels are extensively used for chord members, compression web members, and columns for viaducts. Table XIII gives the dimensions, areas of cross-section, weights per linear foot, and other useful properties of channels varying in depth from 3 to 15 inches. The sizes given in bold-faced type are called **standard channels**, and are kept in stock by rolling mills; the others are rolled to order. Delay is sometimes caused in the work if any but standard channels are ordered, particularly when the rolling mills are busy.

27. Unlike angles, channels are not designated by the width and thickness, but by the depth and weight per linear foot. For instance, a 12-inch 25-pound (12''-25 #) channel is a channel 12 inches deep and weighing 25 pounds per linear foot.

The symbols **C** and **S** are used for *channel* and *channels*, respectively. For example, 3-12''-25 # **S** means three channels each 12 inches deep and each weighing 25 pounds per linear foot.

From Table XIII, any other dimension can be found when the depth and weight are given; thus, the area of cross-section of a 12-inch 25-pound channel is found in column 3 to be

7.35 square inches; the thickness of web, .39 inch, is found in column 4; the width of flange, 3.05 inches, in column 5; etc. Columns 13 to 16 are very useful in working up connections when detailing. Column 17 gives the minimum distance between two channels of the same size, in order that the radius of gyration of the two acting together shall not be less than that given in column 8 for one channel alone.

I BEAMS

28. Dimensions and Properties.—I beams are extensively used to span small openings for both railroads and highways, for the stringers and floorbeams of highway bridges, and for columns supporting elevated railroads. Table XIV gives the dimensions, areas of cross-section, weights per linear foot, and other useful properties of I beams varying in depth from 3 to 24 inches. The sizes given in bold-faced type are called **standard I beams**, and are kept in stock by the rolling mills; the others are rolled to order.

29. I beams, like channels, are designated by the depth and weight per linear foot. For instance, a 20-inch 80-pound (20"-80 #) I beam is a beam 20 inches in depth and weighing 80 pounds per linear foot.

30. Cast-Iron Separators or Spacers.—When I beams are used for small spans under a railroad track, it is sometimes desirable to place two or three beams of the same

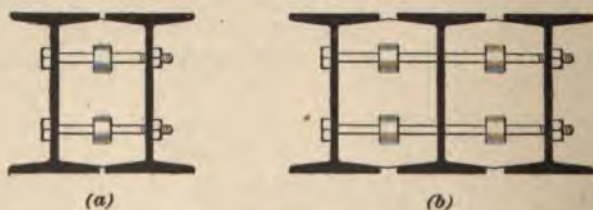


FIG. 2

depth close together under each rail. In such a case, some device is necessary to make the beams work together and maintain a uniform distance apart. This is accomplished by

placing between the **I** beams cast-iron spacers, or separators, and bolting them in place with bolts passing through the spacers and beams. Fig. 2 (*a*) shows a cross-section of two **I** beams connected in this way, and Fig. 2 (*b*) shows three **I** beams connected.

The dimensions and weights of the usual forms of separators, or spacers, are given in Table XV.

OTHER SHAPES

31. Z Bars.—**Z** bars are used in connection with plates for columns, and in floors of bridges when solid steel floors are necessary. Table XVI gives the dimensions, areas of cross-section, weights per linear foot, and other useful properties of **Z** bars varying in depth from 3 to 6 inches. A **Z** bar is designated by giving all the widths and the thickness, the latter being written or named last; as a $3\frac{1}{4}'' \times 5'' \times 3\frac{1}{4}'' \times \frac{5}{16}''$ **Z** bar.

32. I Rails.—It is frequently desirable to know the properties of **I** rails, so that the strength can be determined in cases where they are subject to unusual stresses. Table XVIII gives the weights per yard, dimensions, and properties of American standard rail sections. It is customary to designate rails by the weight per linear yard; thus, a 90-pound rail is a rail that weighs 90 pounds per yard. When second-hand rails are used, proper allowance must be made for the decrease in section due to wear and rust.

REDUCTION OF INCHES TO DECIMALS OF 1 FOOT OR INCH

33. Table XVII is very useful for converting inches into decimal fractions of 1 foot, and for converting into decimal fractions the usual fractional divisions of 1 inch. The last column on the right contains the decimal equivalents of the common fractions in the column immediately preceding it. The numbers in the extreme left column are fractions of an inch; those at the top of the other columns represent

whole inches; and those in the body of the table, decimals of a foot, each of those decimals corresponding to the number of inches denoted by the number at the top of the column, plus the fraction in the left-hand column, horizontally opposite the decimal in question. Thus, the decimal of a foot corresponding to $10\frac{5}{16}$ inches is .8594, found in the column headed 10, and horizontally opposite the fraction $\frac{5}{16}$ in the first column.

EXAMPLES FOR PRACTICE

1. What is the weight of a piece of steel 8.5 feet long, if the area of cross-section is 6.7 square inches? Ans. 193.63 lb.

2. A square steel rod $1\frac{7}{8}$ inches on a side is required to be 15 feet long and to have two upset screw ends. Find, from the tables: (a) the diameter of the upset end; (b) the length of rod before upsetting; (c) the weight of two hexagon nuts for this rod.

$$\text{Ans. } \begin{cases} (a) 2\frac{3}{4} \text{ in.} \\ (b) 15 \text{ ft. } 10\frac{1}{4} \text{ in.} \\ (c) 19.60 \text{ lb.} \end{cases}$$

3. A steel plate has a width of 45 inches and a thickness of $\frac{3}{16}$ inch. Find, from the tables: (a) the area of cross-section of the plate; (b) the weight per linear foot; (c) the greatest length the rolling mills can furnish.

$$\text{Ans. } \begin{cases} (a) 25.31 \text{ sq. in.} \\ (b) 86.06 \text{ lb.} \\ (c) 40 \text{ ft.} \end{cases}$$

4. Find, from the tables, the actual dimensions of angles the nominal dimensions of which are as follows: (a) $3\frac{1}{2}$ in. \times $3\frac{1}{2}$ in. \times $\frac{7}{16}$ in.; (b) 6 in. \times $3\frac{1}{2}$ in. \times $\frac{1}{2}$ in.; (c) 6 in. \times 6 in. \times $\frac{1}{8}$ in.; (d) 7 in. \times $3\frac{1}{2}$ in. \times $\frac{1}{8}$ in.

$$\text{Ans. } \begin{cases} (a) 3\frac{5}{8} \text{ in.} \times 3\frac{5}{8} \text{ in.} \times \frac{7}{16} \text{ in.} \\ (b) 6\frac{1}{8} \text{ in.} \times 3\frac{5}{8} \text{ in.} \times \frac{1}{2} \text{ in.} \\ (c) 6\frac{1}{4} \text{ in.} \times 6\frac{1}{4} \text{ in.} \times \frac{1}{8} \text{ in.} \\ (d) 7\frac{3}{8} \text{ in.} \times 3\frac{1}{8} \text{ in.} \times \frac{1}{8} \text{ in.} \end{cases}$$

5. Find, from the tables, the decimal parts of a foot equivalent to: (a) $1\frac{1}{2}$ inches; (b) $2\frac{3}{4}$ inches; (c) $5\frac{7}{8}$ inches; (d) $11\frac{5}{8}$ inches.

$$\text{Ans. } \begin{cases} (a) .1250 \\ (b) .2292 \\ (c) .4896 \\ (d) .9557 \end{cases}$$

6. Find, from the tables, the decimal parts of an inch equivalent to: (a) $\frac{7}{32}$ inch; (b) $1\frac{9}{16}$ inches; (c) $2\frac{3}{8}$ inches.

$$\text{Ans. } \begin{cases} (a) .2188 \\ (b) 1.5625 \\ (c) 2.9063 \end{cases}$$

RIVETS

34. Introductory.—The rolled shapes that have been described in the preceding articles are connected to one another and to other shapes by means of rivets, those most commonly used in bridge work varying from $\frac{1}{2}$ to 1 inch in diameter. Circular holes $\frac{1}{16}$ inch larger in diameter than the rivets are first punched or drilled in the shapes that are to be connected; the shapes are then placed on one another so that the holes come in line; and heated rivets are inserted in the holes. As soon as a rivet is inserted, and before it has time to cool, the protruding end is hammered or pressed until the rivet completely fills the hole, when a head is formed by tools specially adapted to that purpose.

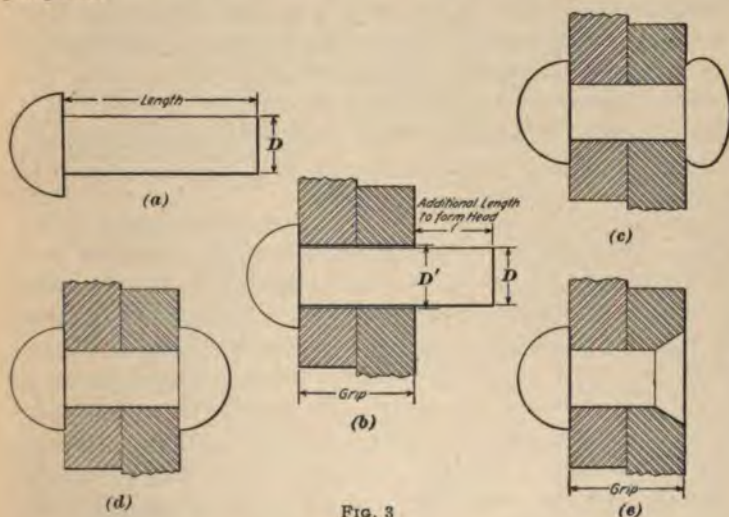


FIG. 3

35. Full and Countersunk Heads.—Each rivet, before driving, has one head, as represented in Fig. 3 (a). In Fig. 3 (b) is shown a rivet inserted in the hole and ready for driving; Fig. 3 (c) shows the rivet when partly driven; and Fig. 3 (d) shows it when completely driven and finished, with the other head formed. This style of rivet is called a

button-head rivet. The rivet represented in Fig. 3 (*d*) is said to have two **full heads**.

When it is objectionable, for any reason, to have the head protrude beyond the surface of the metal, one or both ends of the hole are enlarged to a conical form, and a shorter rivet is used, which is made just long enough to fill the enlarged hole, as represented at the right end of the rivet in Fig. 3 (*c*). Owing to the difficulties of manufacture, such a head will usually project about $\frac{1}{8}$ inch beyond the surface of the metal. This form of head is called a **countersunk head**, and the rivet, a **countersunk rivet**. It is sometimes desirable to use two countersunk heads, one at each end of the rivet.

36. Dimensions of Heads.—Table XIX gives the usual dimensions of button and countersunk heads for rivets from $\frac{1}{2}$ to 1 inch in diameter. In case the heads are too high, and it is not desired to use countersunk heads, they may be flattened by hammering when hot to a height of $\frac{3}{8}$, $\frac{1}{4}$, or $\frac{1}{8}$ inch. If it is desired at any point that a countersunk head should not project at all beyond the surface of the metal, it must be planed or **chipped** off with a chisel.

37. Dimensions and Weights.—The distance from the under side of the head to the end of the rivet before driving, as represented in Fig. 3 (*a*), is called the **length**; the cylindrical portion of the rivet is called the **shank**; and the thickness of metal between the heads, as represented in Fig. 3 (*b*), is called the **grip**. Table XX gives the additional length of shank required to form a head for rivets from $\frac{1}{2}$ to 1 inch in diameter and for grips of from $\frac{1}{2}$ inch to 6 inches. Part of this additional length is utilized in filling the rivet hole whose diameter is D' , Fig. 3 (*b*), and the remainder in forming the head. On account of the necessity of filling the hole, a greater additional length is required for long grips than for short grips.

To obtain the length of rivet required for a given grip and diameter of rivet, add to the grip the additional length given in Table XX for that rivet corresponding to that grip.

For example, if a $\frac{7}{8}$ -inch rivet has a grip of $3\frac{1}{2}$ inches, then, to form a button head, since the additional length corresponding to a grip of $3\frac{1}{2}$ inches for a $\frac{7}{8}$ -inch rivet is given in Table XX as $1\frac{3}{4}$ inches, the length before driving should be $3\frac{1}{2} + 1\frac{3}{4} = 5\frac{1}{4}$ inches, and to form a countersunk head, $3\frac{1}{2} + 1 = 4\frac{1}{2}$ inches.

The weights of button heads for rivets from $\frac{1}{2}$ to 1 inch in diameter are given in Table XXI. The weights of shanks can be found by the use of Table I.

EXAMPLE.—To find the weight of 125 rivets $\frac{7}{8}$ inch in diameter, having a grip of $2\frac{5}{8}$ inches, if both ends are to have button heads.

SOLUTION.—The additional length of shank required to form a button head on a $\frac{7}{8}$ -in. rivet having a grip of $2\frac{5}{8}$ in. is given in Table XX as $1\frac{5}{8}$ in.; then, the length before driving will be $2\frac{5}{8} + 1\frac{5}{8} = 4\frac{1}{4}$ in., or, by Table XVII, .3542 ft. Table I gives the weight per linear foot of a round rod $\frac{7}{8}$ in. in diameter as 2.04 lb.; then, the weight of the rivet shanks, since there are 125 rivets, is

$$125 \times .3542 \times 2.04 = 90.32 \text{ lb.}$$

Table XXI gives the weight of 100 button heads for $\frac{7}{8}$ -in. rivets as 16.7 lb.; then, the weight of 125 heads is $\frac{125}{100} \times 16.7 = 20.88$ lb. The total weight of the rivets is

$$90.32 + 20.88 = 111.20 \text{ lb. Ans.}$$

38. Conventional Signs for Riveting.—Those rivets that are driven where the bridge is manufactured are called **shop rivets**; those that are driven where the bridge is erected, **field rivets**. Table XXIII (*a*) and (*b*) shows the conventional signs used in preparing drawings to indicate where the rivet is to be driven and what type of head is desired. By **this side**, **outside**, or **near side** is meant the surface shown uppermost on the drawing, or the upper side; by **other side**, **inside**, or **far side** is meant the surface opposite that shown uppermost, or the under side. Both of these systems are standard in different offices; that shown in Table XXIII (*a*) is the **American Bridge Company's standard**; that shown in Table XXIII (*b*) is called the **Osborne standard**.

39. Rivet Pitch.—The distance between the centers of two consecutive rivets in the same row is called the **pitch**

of the rivets. If there is one row, as in Fig. 4 (a), the pitch is the distance between two rivets in the same row; if there are two rows, as represented in Fig. 4 (b), the pitch is the distance between two consecutive rivets in alternate rows,

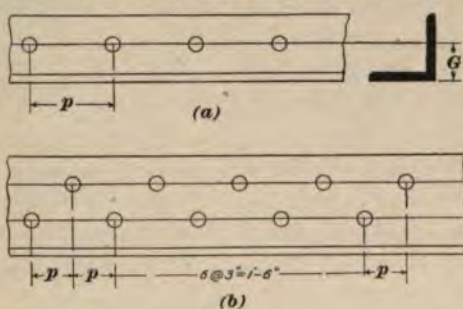


FIG. 4

measured parallel to the gauge lines. The pitch is marked p in Fig. 4.

In spacing rivets along gauge lines, it is customary to make the pitch uniform, that is, to have as many spaces of the same length together

as is convenient. In specifying the number of spaces, pitch, etc., it is customary to write first the number of equal spaces, then the pitch, and finally the distance covered by those equal spaces [see Fig. 4 (b)].

Table XXII gives the total distances covered by equal spaces from $1\frac{1}{2}$ to 6 inches, and from 2 to 32 in number.

EXAMPLES FOR PRACTICE

1. Find the total length of shank required to form a button head for: (a) a $\frac{3}{4}$ -inch rivet having a grip of 2 inches; (b) a $\frac{7}{8}$ -inch rivet having a grip of $3\frac{1}{8}$ inches; (c) a 1-inch rivet having a grip of $3\frac{1}{2}$ inches.

$$\text{Ans. } \begin{cases} (a) & 3\frac{1}{2} \text{ in.} \\ (b) & 4\frac{7}{8} \text{ in.} \\ (c) & 5\frac{3}{8} \text{ in.} \end{cases}$$

2. Find the total length of shank required to form one countersunk head for: (a) a $\frac{3}{4}$ -inch rivet having a grip of $2\frac{1}{4}$ inches; (b) a $\frac{7}{8}$ -inch rivet having a grip of $2\frac{3}{4}$ inches.

$$\text{Ans. } \begin{cases} (a) & 3\frac{1}{8} \text{ in.} \\ (b) & 3\frac{5}{8} \text{ in.} \end{cases}$$

3. Find the weight of 245 rivets $\frac{3}{4}$ inch in diameter and having a grip of $2\frac{1}{2}$ inches: (a) if both ends have button heads; (b) if one end has a countersunk head.

$$\text{Ans. } \begin{cases} (a) & 141 \text{ lb.} \\ (b) & 122 \text{ lb.} \end{cases}$$

4. Find, from Table XXII, the total distances covered by the following numbers of equal spaces: (a) eleven spaces at $2\frac{7}{8}$ inches;

(b) seventeen spaces at $3\frac{3}{8}$ inches; (c) twenty spaces at $3\frac{1}{2}$ inches;
 (d) twenty-seven spaces at $4\frac{3}{4}$ inches; (e) thirty-one spaces at $3\frac{1}{2}$ inches.

$$\text{Ans.} \left\{ \begin{array}{l} (a) \text{ 2 ft. } 7\frac{5}{8} \text{ in.} \\ (b) \text{ 4 ft. } 9\frac{3}{8} \text{ in.} \\ (c) \text{ 5 ft. } 2\frac{1}{2} \text{ in.} \\ (d) \text{ 10 ft. } 8\frac{1}{4} \text{ in.} \\ (e) \text{ 9 ft. } \frac{1}{2} \text{ in.} \end{array} \right.$$

BUILT-UP SHAPES

40. The simple shapes that have been explained and tabulated in the preceding pages are seldom used alone for bridge members; they are usually employed in connection with other shapes, the several parts being thoroughly connected and riveted together so as to act as a single shape. Such combinations are called **built-up**, or **compound**, **shapes**, the same names being applied to members formed with them. The methods of connecting the different parts of a built-up shape will be treated later.

TWO ANGLES BACK TO BACK

41. Angles are frequently used in pairs, the backs being placed either together or parallel to each other and a short distance apart. When two angles are used for a compression member, it is necessary to know the radii of gyration of the shape about two axes passing through the center of gravity, one at right angles, and the other parallel to the adjacent backs of the angles. Tables XXIV, XXV, and XXVI give the radii of gyration about the two axes passing through the center of gravity of the section, for pairs of angles when placed in contact and also at distances of $\frac{1}{4}$, $\frac{1}{2}$, and $\frac{3}{4}$ inch apart, respectively. Only values for the maximum and minimum thicknesses of angles are given; those for any intermediate thickness can be found by interpolation.

EXAMPLE.—To find, from Table XXIV, the radius of gyration about an axis parallel to the adjacent backs of two $6'' \times 6'' \times \frac{5}{8}''$ angles placed $\frac{1}{4}$ inch apart.

SOLUTION.—It is found, from the table, that the required radius of gyration for two $6'' \times 6'' \times \frac{5}{8}''$ angles is 2.58 in., and for two $6'' \times 6'' \times 1''$

angles, 2.68 in. The difference in the thickness of these angles is $\frac{5}{8}$, or $\frac{10}{16}$, in., and the difference between the radii of gyration is $2.68 - 2.58 = .10$ in., or $\frac{.10}{10} = .01$ for each $\frac{1}{16}$ inch increase in the thickness of the angle. The difference between the minimum thickness, $\frac{3}{8}$ inch, and that of the angles that are to be used, $\frac{5}{8}$ inch, is $\frac{1}{4}$; then, the increase in the radius of gyration is $.01 \times 4 = .04$, and the required radius is $2.58 + .04 = 2.62$ in. Ans.

OTHER BUILT-UP SHAPES

42. Standard Forms.—Fig. 5 shows the forms of built-up shapes in most common use. Forms (a) to (c) are composed of plates, angles, and channels, arranged as shown, and are used for upper chords and end posts. The vertical plates are called the **web-plates**; the angles that connect to their edges, the **flanges**; and the horizontal plate at the top, the **cover-plate**.

Forms (f) to (l) are composed of simple shapes, arranged symmetrically as shown, and are used for lower chords and web members of trusses, and for columns in trestle bents. Forms (m) and (n) are composed of channels and I beams arranged symmetrically, and are used, to some extent, for columns for elevated railways.

43. Properties.—When a built-up shape is used for a compression member, it is necessary to know the area of cross-section, and also the radii of gyration about two axes passing through the center of gravity of the cross-section, one of which is parallel and the other at right angles to the webs. The area of cross-section can be readily found by adding together the areas of cross-section of the different parts of which the shape is composed. The location of the center of gravity can be determined by the principles of statics, as explained in *Analytic Statics*, Part 2, and the radii of gyration can be found as explained in *Strength of Materials*, Part 2. There is so great a variety of built-up shapes that it is difficult to prepare tables covering all the cases that are likely to occur in practice. In any particular case, the required quantities must be determined by calculation.

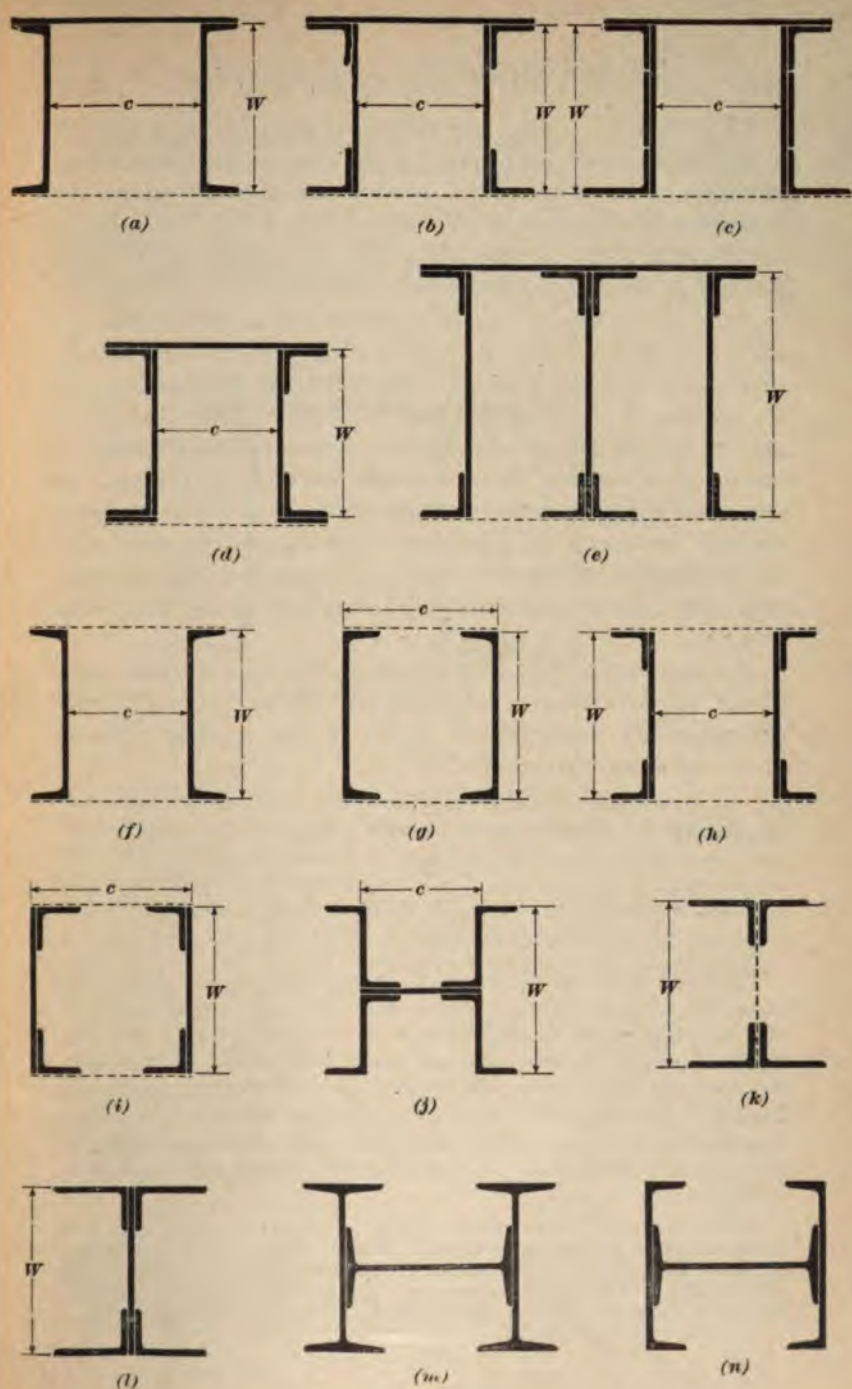


FIG. 5

44. In order to find the radius of gyration of a section with reference to any axis, it is necessary to know the moment of inertia of the section about that axis. As explained in *Strength of Materials*, Part 2, the moment of inertia of a plane surface about an axis in the same plane and not passing through the center of gravity of the surface is equal to the sum of the moment of inertia of the surface about an axis through its center of gravity parallel to the given axis, and the product of the area of the surface and the square of the perpendicular distance from its center of gravity to the given axis. If I_1 represents the moment of inertia of a surface about an axis not passing through its center of gravity, I_s its moment of inertia about an axis through its center of gravity and parallel to the first axis, A_1 the area of the surface, and y_1 the perpendicular distance from the center of gravity of the surface to the first axis, then,

$$I_1 = I_s + A_1 y_1^2$$

If I represents the moment of inertia of a surface composed of several smaller surfaces, and I_1', I_1'', I_1''' , etc. represent the moments of inertia of the smaller surfaces about the same axis, then

$$I = I_1' + I_1'' + I_1''' + \dots$$

or, letting ΣI_1 represent the sum $I_1' + I_1'' + I_1''' + \dots$

$$I = \Sigma I_1 \quad (1)$$

Also, substituting for I_1 its value $I_s + A_1 y_1^2$,

$$I = \Sigma (I_s + A_1 y_1^2) \quad (2)$$

EXAMPLE 1.—A built-up section, arranged as shown in Fig. 6, is composed of the following simple sections: one cover-plate 26 in. $\times \frac{1}{2}$ in.; two upper flange angles 4 in. $\times 4$ in. $\times \frac{5}{8}$ in.; two web-plates 20 in. $\times \frac{5}{8}$ in.; and two lower flange angles 6 in. $\times 4$ in. $\times \frac{5}{8}$ in. To find: (a) the area of the section, and the location of the two axes $Y'Y$, parallel, and $X'X$, perpendicular, to the webs, and passing through the center of gravity of the section; (b) the moment of inertia about each of these axes; (c) the radius of gyration referred to each of these axes.

NOTE.—The vertical distance from the top surface of the upper flange angle to the bottom surface of the lower flange angle, $20\frac{1}{4}$ inches in Fig. 6, is called the **distance back to back of angles**, and is made $\frac{1}{4}$ inch greater than the width of the web-plate, to allow for irregularities in the latter. The horizontal distance between the web-plates, 15 inches in Fig. 6, is called the **clear distance between webs**.

SOLUTION.—(a) As the section is symmetrical about the axis $Y'Y$, half way between the webs, the center of gravity lies on this axis. As the section is not symmetrical about any axis at right angles to $Y'Y$, the location of the axis $X'X$, perpendicular to $Y'Y$, must be found by calculation. The areas of cross-section and the location of centers of gravity of the simple sections will be taken from the tables. As

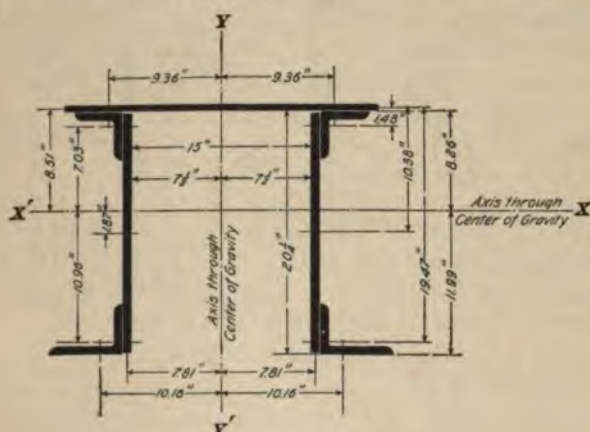


FIG. 6

moments may be taken about any axis, it will be convenient to take them about an axis passing through the center of the cover-plate. The lever arms are shown at the right-hand side in Fig. 6. For example, the distance from the back of a $4'' \times 4'' \times \frac{5}{8}''$ angle to the center of gravity is given in Table IX as 1.23 in.; then, its lever arm is equal to the sum of 1.23 and one-half the thickness of the cover-plate, or $1.23 + .25 = 1.48$ in.; similarly for the other simple sections. The calculation is as follows:

SECTIONS	AREA FROM TABLES SQUARE INCHES	LEVER ARM	MOMENTS
One plate 26 in. $\times \frac{1}{2}$ in.	= 13.00	0	0
Two angles 4 in. \times 4 in. $\times \frac{5}{8}$ in.	$2 \times 4.61 =$	9.22	1.48
Two plates 20 in. $\times \frac{5}{8}$ in.	$2 \times 12.5 =$	25.00	10.38
Two angles 6 in. \times 4 in. $\times \frac{5}{8}$ in.	$2 \times 5.86 =$	11.72	19.47
Total,	58.94		501.34

Dividing the total moment (501.34) by the entire area (58.94) of the section gives $501.34 \div 58.94 = 8.51$ in. from the center of the cover-plate to the axis $X'X$. Then, $8.51 - .25 = 8.26$ in. is the distance from the axis $X'X$ to the back of the upper flange angles. Ans.

(b) The moment of inertia about the axis $Y'Y$ will first be found. The lever arms y_1 are plainly shown in Fig. 6.

ONE PLATE 26 IN. $\times \frac{1}{2}$ IN.

$$I_2 \text{ (Table VIII)} = 2 \times 6.66 \dots\dots\dots = 732.3$$

$$A_1 y_1^2 = 13 \times 0 \times 0 \dots\dots\dots = 0$$

TWO ANGLES 4 IN. \times 4 IN. $\times \frac{5}{8}$ IN.

$$I_2 \text{ (Table IX)} = 2 \times 6.66 \dots\dots\dots = 13.3$$

$$A_1 y_1^2 = 9.22 \times 9.36^2 \dots\dots\dots = 807.8$$

TWO PLATES 20 IN. $\times \frac{5}{8}$ IN.

$$I_2 = 2 \times \frac{1}{12} \times 20 \times \left(\frac{5}{8}\right)^3 \dots\dots\dots = .8$$

$$A_1 y_1^2 = 25 \times 7.81^2 \dots\dots\dots = 1524.9$$

TWO ANGLES 6 IN. \times 4 IN. $\times \frac{5}{8}$ IN.

$$I_2 \text{ (Table X)} = 2 \times 21.07 \dots\dots\dots = 42.1$$

$$A_1 y_1^2 = 11.72 \times 10.16^2 \dots\dots\dots = 1209.8$$

Moment of inertia about $Y'Y$ $\dots\dots\dots = 4331.0$ Ans.

The moment of inertia about the axis $X'X$ will now be found. The lever arms y_1 are shown at the left-hand side in Fig. 6.

ONE PLATE 26 IN. $\times \frac{1}{2}$ IN.

$$I_2 = \frac{1}{12} \times 26 \times \left(\frac{1}{2}\right)^3 \dots\dots\dots = .3$$

$$A_1 y_1^2 = 13 \times 8.51^2 \dots\dots\dots = 941.5$$

TWO ANGLES 4 IN. \times 4 IN. $\times \frac{5}{8}$ IN.

$$I_2 \text{ (Table IX)} = 2 \times 6.66 \dots\dots\dots = 13.3$$

$$A_1 y_1^2 = 9.22 \times 7.03^2 \dots\dots\dots = 455.7$$

TWO PLATES 20 IN. $\times \frac{5}{8}$ IN.

$$I_2 \text{ (Table VIII)} = 2 \times 416.67 \dots\dots\dots = 833.3$$

$$A_1 y_1^2 = 25.0 \times 1.87^2 \dots\dots\dots = 87.4$$

TWO ANGLES 6 IN. \times 4 IN. $\times \frac{5}{8}$ IN.

$$I_2 \text{ (Table X)} = 2 \times 7.52 \dots\dots\dots = 15.0$$

$$A_1 y_1^2 = 11.72 \times 10.96^2 \dots\dots\dots = 1407.8$$

Moment of inertia about $X'X$ $\dots\dots\dots = 3754.3$ Ans.

(c) The radius of gyration is found by the formula $r = \sqrt{\frac{I}{A}}$. If r_y and r_x are the radii of gyration about $Y'Y$ and $X'X$, respectively, this formula gives

$$r_y = \sqrt{\frac{4,331.0}{58.94}} = \sqrt{73.48} = 8.57 \text{ in. Ans.}$$

$$r_x = \sqrt{\frac{3,754.3}{58.94}} = \sqrt{63.70} = 7.98 \text{ in. Ans.}$$

EXAMPLE 2.—A built-up section is composed of two web-plates 15 in. $\times \frac{1}{2}$ in., and four flange angles 4 in. \times 4 in. $\times \frac{1}{2}$ in., arranged as shown in Fig. 7. To find: (a) the area of the entire cross-section;

(b) the location of the axes passing through the center of gravity; (c) the moment of inertia with respect to each axis; (d) the radius of gyration with respect to each axis.

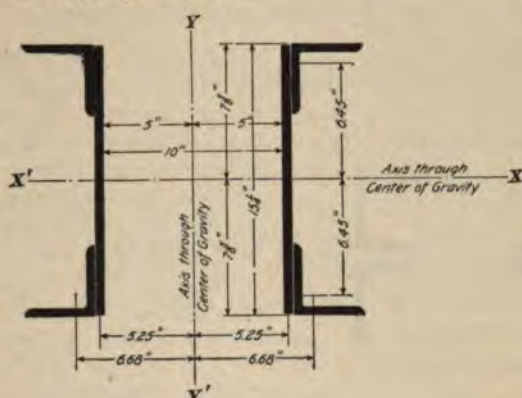


FIG. 7

SOLUTION.—(a) The area of two plates $15 \text{ in.} \times \frac{1}{2} \text{ in.}$ is 15 sq. in.; the area of four angles $4 \text{ in.} \times 4 \text{ in.} \times \frac{1}{2} \text{ in.}$ is $4 \times 3.75 = 15 \text{ sq. in.}$ Then, the area of the entire section is

$$15 + 15 = 30 \text{ sq. in.} \quad \text{Ans.}$$

(b) As the section is symmetrical, the axis $Y'Y$ is 5 in. from the inside surface of each web, and the axis $X'X$ is $7\frac{5}{8}$ in. from the backs of the flange angles. Ans.

(c) The distances from $X'X$ to the center of gravity of the various angles is shown at the right in Fig. 7.

TWO PLATES 15 IN. \times $\frac{1}{2}$ IN.

$$I_2 \text{ (Table VIII)} = 2 \times 140.63 \dots\dots\dots = 281.3$$

FOUR ANGLES 4 IN. \times 4 IN. \times $\frac{1}{8}$ IN.

$$I_2 \text{ (Table IX)} = 4 \times 5.56 \dots\dots\dots = 22.2$$

$$A_1 y_1^2 = 15 \times 6.45^2 \dots\dots\dots = 624.0$$

Moment of inertia about $X'X$ = 927.5 Ans.

The distance from $Y'Y$ to the center of gravity of the angles and webs is shown on the lower side in Fig. 7.

TWO PLATES 15 IN. \times $\frac{1}{2}$ IN.

$$I_2 = \frac{2}{1.2} \times 15 \times \left(\frac{1}{2}\right)^2 \dots \dots \dots = .3$$

$$A_1 y_1^2 = 15 \times 5,25^2 \dots\dots\dots = 413,4$$

FOUR ANGLES 4 IN. \times 4 IN. \times $\frac{1}{2}$ IN.

$$I_s \text{ (Table IX)} = 4 \times 5.56 \dots\dots\dots = 22.2$$

$$A_1 y_1^2 = 15 \times 6.68^2 \dots\dots\dots = 669.3$$

Moment of inertia about $Y'Y$ = 1105.2 Ans.

$$(d) \quad r_x = \sqrt{\frac{I}{A}} = \sqrt{\frac{927.5}{30}} = \sqrt{30.92} = 5.55 \text{ in. Ans.}$$

$$r_y = \sqrt{\frac{1,105.3}{30}} = \sqrt{36.84} = 6.07 \text{ in. Ans.}$$

EXAMPLE 3.—A built-up section is composed of four $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ angles arranged as shown in Fig. 8. To find: (a) the area of the cross-section; (b) the moments of inertia with respect to axes through the center of gravity; (c) the radii of gyration with respect to the same axes.

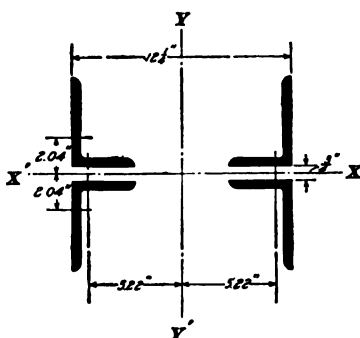


FIG. 8

SOLUTION.—(a) The area of the cross-section is equal to

$$4 \times 4 = 16 \text{ sq. in. (Table X).}$$

Ans.

(b) As the section is symmetrical, its center of gravity is its center of symmetry.

The moment of inertia with respect to the axis $X'X$ is as follows:

FOUR ANGLES 5 IN. \times $3\frac{1}{2}$ IN. \times $\frac{1}{2}$ IN.

$$I_x = 4 \times 9.99 \dots\dots\dots = 39.96$$

$$A_1 y_1^2 = 16 \times 2.04^2 \dots\dots\dots = 66.59$$

106.55 Ans.

The moment of inertia with respect to the axis $Y'Y$ is as follows:

FOUR ANGLES 5 IN. \times $3\frac{1}{2}$ IN. \times $\frac{1}{2}$ IN.

$$I_y = 4 \times 4.05 \dots\dots\dots = 16.20$$

$$A_1 y_1^2 = 16 \times 5.22^2 \dots\dots\dots = 435.97$$

452.17 Ans.

$$(c) \quad r_x = \sqrt{\frac{106.55}{16}} = \sqrt{6.66} = 2.58 \text{ in. Ans.}$$

$$r_y = \sqrt{\frac{452.17}{16}} = \sqrt{28.26} = 5.32 \text{ in. Ans.}$$

The radius of gyration with respect to the axis $X'X$ could in this example have been found from Table XXVI, as the radius of gyration of the four angles with respect to this axis is the same as that of two angles. In Table XXVI, the radius of gyration for two angles with the short legs parallel and $\frac{3}{4}$ in. apart is given in the column headed r_x ; that for two angles 5 in. \times $3\frac{1}{2}$ in. \times $\frac{1}{2}$ in. is 2.54 in.; and that for two angles 5 in. \times $3\frac{1}{2}$ in. \times $\frac{1}{2}$ in. is 2.65 in. The difference in thickness of these angles is $\frac{9}{16}$ in., and the difference between their radii of gyration is .11. The difference in thickness between $\frac{5}{16}$ and $\frac{1}{2}$ is $\frac{3}{16}$;

then the difference between the radii of gyration is $\frac{3}{8}$ of $.11 = .04$, which added to 2.54 gives 2.58 in., as before.

45. Remarks on Example 1.—In example 1 of Art. 44, it will be seen that the radius of gyration r_y with respect to $Y'Y$ is greater than the radius r_x with respect to the axis $X'X$; also, that the clear distance between webs (c , Fig. 5), is approximately equal to three-fourths of the distance W back to back of the flange angles. It can be shown that, for the sections (a), (b), (c), and (d), Fig. 5, r_y is invariably greater than r_x when c is greater than $\frac{3}{4}W$. Therefore, if, as is usually the case, only the least radius of gyration is required, it is unnecessary to compute r_y if c is greater than $\frac{3}{4}W$. This relation does not hold for section (e), for which both radii of gyration must be computed in order to find the smaller. It will also be seen that r_x is approximately equal to $\frac{3}{8}W$, and it may be shown that this relation is true in almost all cases for the forms (a), (b), (c), (d), and (e). This is convenient in approximate calculations, and is made use of in designing before the final cross-section of a member has been found.

46. Remarks on Example 2.—In example 2 of Art. 44, it will be seen that r_y is greater than r_x , and that c is approximately equal to $\frac{3}{8}W$. It may be shown that, if only the least radius of gyration is required, it is unnecessary to compute r_y if c is greater than $\frac{3}{8}W$, for the forms (f) and (h), Fig. 5, and, if c is greater than $\frac{7}{8}W$, for the forms (g) and (i). For approximate calculation, the least radius of gyration for the forms (f), (g), (h), and (i) will be r_x , and may be taken equal to $\frac{1}{8}W$ if c and W bear the proper relation, that is, if c is greater than $\frac{3}{8}W$, for the forms (f) and (h), and greater than $\frac{7}{8}W$ for the forms (g) and (i).

47. It is usually specified that the ratio $\frac{l}{r}$ of the unsupported length l of a compression member to its least radius of gyration r shall not exceed a certain number, generally from 80 to 120. The approximate relations stated in the two preceding articles afford a ready means of calculating

the smallest allowable width W and clear distance c between webs of compression members when the greatest allowable value of $\frac{l}{r}$ is known. For example, if it is specified that $\frac{l}{r}$ shall not exceed 80, then, for the maximum allowable value of the ratio, $\frac{l}{r} = 80$, and $r = \frac{l}{80}$. For a member whose length is 25 feet, or 300 inches, $r = 300 \div 80 = 3.75$ inches. If the form (I), Fig. 5, is used, r is approximately equal to $\frac{1}{3}W$; and, as r is 3.75 inches, $W = 3 \times 3.75 = 11.25$ inches, and $c = \frac{2}{3} \times 11.25 = 7.5$ inches. In this case, W would probably be made 12 inches or more, and c , 8 inches or more, according to other details.

48. In the forms (j), (m), and (n), r_x is invariably less than r_y , and in the forms (k) and (l), r_y is invariably less than r_x ; only the former need be computed if only the least radius of gyration is required.

49. The forms shown in Fig. 5 are frequently modified by the addition of other simple sections, principally plates. The methods of calculation are precisely the same as described in the preceding articles, and should present no further difficulty. It should be remembered that, in finding the radius of gyration of a built-up section, both values should be found if there is the least doubt as to which is the smaller.

50. **Location of Center Line.**—As already explained, when the stresses in the members of a truss are to be determined, each member is represented by a line. These lines are called the **center lines** of the members. For vertical and inclined members, the center line passes through the center of gravity of the section; for chord members, the center line lies close to the center of gravity, but does not pass through it. The distance from the center line of a member to the center of gravity of the cross-section of the member is called the **eccentricity** of the member. The center line of a built-up chord member is located *below the center of gravity in compression members*, and *above the center of*

gravity in tension members, the eccentricity being such that the bending moment on the member due to the eccentricity of the stress shall be equal and opposite to that due to the weight of the member.

The eccentricity e , for chord members, is found by dividing the bending moment M_w of the member due to its own weight by the total stress S in the member; that is,

$$e = \frac{M_w}{S} \quad (1)$$

If W and l are, respectively, the weight and length of the member, then (see *Strength of Materials*, Part 1),

$$M_w = \frac{Wl}{8}$$

This value in formula 1 gives

$$e = \frac{Wl}{8S} \quad (2)$$

If the weight of the member per foot is denoted by w , then $W = wl$, and, therefore,

$$e = \frac{wl^2}{8S} \quad (3)$$

EXAMPLE.—The gross section of a member whose length is 20 feet is 20 square inches. To find the eccentricity of the center line if the total stress is 200,000 pounds.

SOLUTION.—As the gross section of the member is 20 sq. in., its weight per linear foot is $20 \times 3.4 = 68$ lb. (Art. 15), and, therefore, applying formula 3,

$$e = \frac{68 \times 20^2}{8 \times 200,000} = .017 \text{ ft.} = .204 \text{ in.} \quad \text{Ans.}$$

RIVETING, LATTICE BARS, AND TIE-PLATES

51. Riveting.—Wherever two portions of a built-up member come in contact, a row of rivets is driven, as represented in Fig. 9, in which (*a*) is the top view or plan, (*b*) the side elevation, (*c*) a horizontal section qq and bottom view, and (*d*) a vertical section gg , of a short piece of an upper chord; and in Fig. 10, which is a short piece of a compression web member, (*a*), (*b*), (*c*), and (*d*), representing the same views as in Fig. 9—that is, a top view, side elevation, longitudinal

tion and bottom view and cross-section, respectively. The vertical legs of the flange angles are riveted to the webs with rivets located on the gauge lines ee , Fig. 9 (b) and Fig. 10 (b), of the angles; the cover-plate is riveted to the

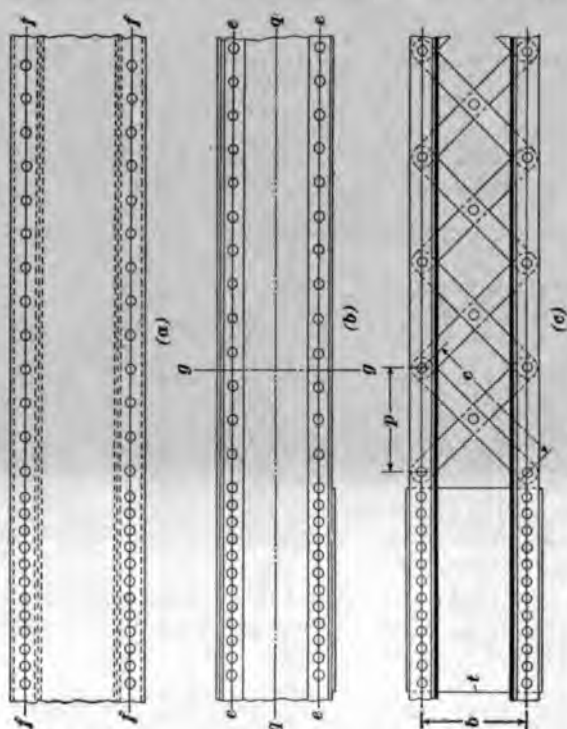
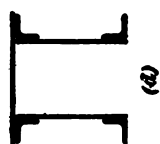


FIG. 9



horizontal legs, frequently called the **outstanding legs**, of the upper flange angles with rivets located on the gauge lines ff of the angles, Fig. 9 (a) and Fig. 10 (a). For a short distance at each end, near where the member connects at the joint, as shown at the left end of Figs. 9 and 10, the

rivets are spaced about 3 inches apart; for the remainder of the distance, they are usually spaced 6 inches apart. The rivets in the horizontal legs of the flange angles are usually located half way between those in the vertical legs, to facili-

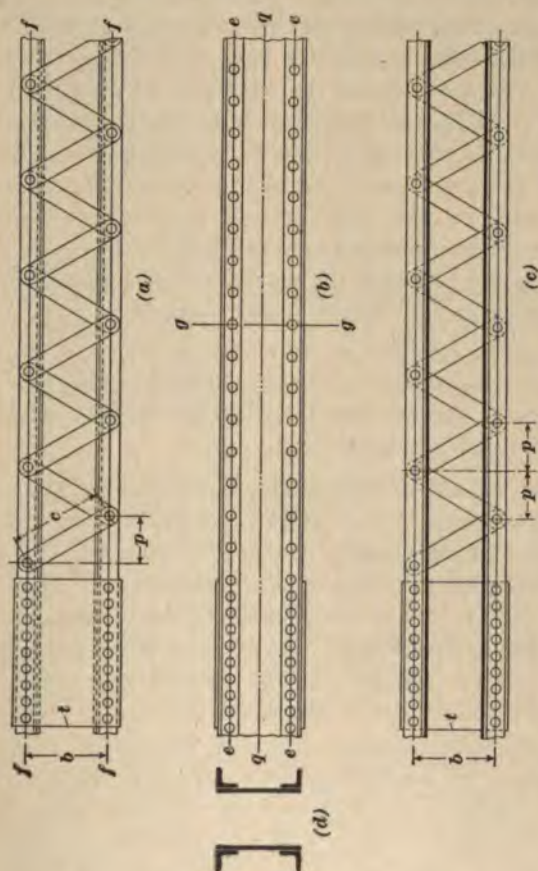


FIG. 10

tate driving them; this is called **staggering the rivets** in the two legs.

52. Lattice Bars.—The open sides of built-up members are connected to each other by diagonal bars whose ends are riveted to the outstanding legs of the flanges, as shown

in Fig. 9 (*c*), and in Fig. 10 (*a*) and (*c*). These diagonals are called **lattice bars**. In Fig. 9 (*c*), the bars cross each other and are riveted at the intersection; this construction is called **double latticing**. The method of latticing shown in Fig. 10 (*a*) and (*c*) is called **single latticing**. In double latticing, the flange is divided into a number of equal spaces of such length that the lattice bars will make an angle of about 45° with the axis of the member; in single latticing, the flange is divided into a number of equal spaces of such length that the lattice bar will make an angle of about 60° with the axis of the member. The longitudinal distance, or distance parallel to the axis of the member, between the connections of the ends of a lattice bar is called the **pitch**.

The distance c between the centers of the holes in the ends of a lattice bar can be found by the formula

$$c = \sqrt{p^2 + b^2}$$

in which b = transverse distance between gauge lines of rivets that connect lattice bars to flanges;
 p = pitch of latticing.

53. Lattice bars are made from flat bars from 2 to 4 inches in width and from $\frac{1}{4}$ to $\frac{5}{8}$ inch in thickness; the ends are rounded to eliminate sharp protruding corners. It is usually required that the thickness of lattice bars shall be not less than one-fortieth of the distance between the centers of the rivets that connect them to the flanges for single latticing, or one-sixtieth for double latticing. No formula can be given from which to determine when to use single and when double latticing; nor is there any formula for the size of lattice bars. These matters are controlled largely by practice, and depend to a great extent on the width of the member. An approximate rule that agrees fairly well with practice is to use single latticing in all cases where the bars are less than about 15 inches in length, and double latticing in all other cases. The usual locations of the latticing are shown by dotted lines in Fig. 5.

54. Tie-Plates.—For a short distance at each end of a built-up member, near where it is connected at the joints, the

latticing is omitted and the flanges are connected by **tie-plates**, as represented at *t*, Fig. 9 (*c*) and Fig. 10 (*a*) and (*c*). These plates are placed as close to the ends of the members as the connections will permit, and are usually made from one to two times as long as they are wide, and from $\frac{1}{4}$ to $\frac{1}{2}$ inch in thickness. The plates are connected to the flanges by rivets spaced from 3 to 4 inches apart. The sizes and lengths of tie-plates depend, to a great extent, on practice. They are used in connection with lattice bars to keep the stress evenly distributed between the sides of the member.

GROSS AND NET SECTIONS

55. Definition.—The areas of cross-sections that have been used in the preceding articles are the **gross areas**, commonly called **gross sections**. When a portion of the sectional area of a built-up member is cut away for any purpose, the section is decreased, and the remainder is called the **net section**. In the case of rivet holes, as they are completely filled with rivets, the member offers about as much resistance to compression after the rivets are driven as before the holes were punched, and so the gross area is available in resisting compression, except where there are pinholes and bolt holes; as these are not completely filled, their areas must be deducted. Rivets, bolts, and pins cannot transmit tension from one side of the hole to the other, however, and so only the net section is available in resisting tension.

56. Deduction of Holes.—In finding the net section of a member, it is customary to deduct from the gross section the area of cross-section of the largest number of holes that can be cut by a plane at right angles to the axis of the member. As the holes are $\frac{1}{8}$ inch larger in diameter than the rivets, and as the metal immediately surrounding the holes is somewhat injured by punching, it is common practice to deduct for each hole the area of cross-section of a hole $\frac{1}{8}$ inch larger in diameter than the rivet. Table XXVII gives the area of section to be deducted for each hole for

rivets from $\frac{1}{2}$ to 1 inch in diameter and material $\frac{1}{4}$ inch to $1\frac{1}{2}$ inches in thickness; the areas are found from the formula

$$A = t(d + \frac{1}{8} \text{ inch})$$

in which

A = area, in square inches;

d = diameter of rivet;

t = thickness of metal.

For example, the plane gg , Fig. 10, cuts four rivets—two shown in the figure and the two corresponding on the other side—each of which passes through a web and a flange angle. Then, in order to get the net section, there must be deducted from the gross section the areas of four holes in the webs and four holes in the angles. Suppose that the webs are $\frac{5}{8}$ inch thick, angles $\frac{1}{2}$ inch thick, and rivets $\frac{7}{8}$ inch in diameter. From Table XXVII, it is seen that, for a $\frac{7}{8}$ -inch rivet in material $\frac{1}{2}$ inch thick, there must be deducted .50 square inch, and in material $\frac{5}{8}$ inch thick, .625 square inch; then, the total area to be deducted in this case is

$$(4 \times .50) + (4 \times .625) = 4.5 \text{ square inches}$$

If the gross section is 35 square inches, the net section will be $35 - 4.5 = 30.5$ square inches.

57. It is evident also that, if a rivet lies very close to the plane that cuts the rivets just referred to, the section shall be still further decreased. For such cases, the following rule has been found in actual practice to give fairly accurate results for rivets $\frac{7}{8}$ inch in diameter or larger:

Rule.—*To find the net section of a built-up member when the gross section is known, consider the member cut by a plane at right angles to the member in such a position that it will pass through the centers of the largest number of rivets; deduct from the gross section the area of cross-section of one hole for each rivet whose center lies within $\frac{3}{4}$ inch of the plane, and a proportionate part of one hole for each rivet whose center lies within $2\frac{3}{4}$ inches.*

For example, if the center of a rivet is $\frac{3}{4}$ inch from the plane, the entire area for one hole is to be deducted; if $1\frac{3}{4}$ inches, one-half the area for one hole; if $2\frac{1}{4}$ inches, one-quarter the area; and if $2\frac{3}{4}$ inches, no deduction is to be made. For rivets $\frac{3}{4}$ inch in diameter or smaller, the

foregoing limits, $\frac{3}{4}$ inch and $2\frac{3}{4}$ inches, should be changed to $\frac{1}{2}$ inch and 2 inches, respectively.

EXAMPLE.—A tension member consisting of one plate 12 in. \times $\frac{9}{16}$ in. has its section reduced by the group of $\frac{7}{8}$ -inch rivets shown in Fig. 11. Find: (a) the gross section of the plate; (b) the net section.

SOLUTION.—(a) The gross-section of a plate 12 in. \times $\frac{9}{16}$ in. (Table VI) is 6.75 sq. in.

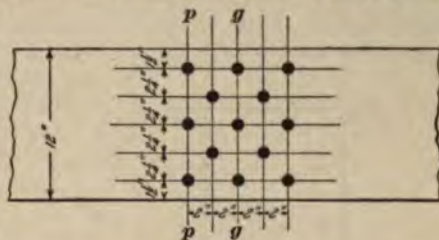


FIG. 11

(b) Consider the plate cut by the plane $p p$ that passes through the centers of three rivets. The area to be deducted for one $\frac{7}{8}$ -inch rivet in material $\frac{9}{16}$ inch in thickness is, from Table XXVII, .563 sq. in.; as there are three rivets cut by the plane, the deduction for them is $3 \times .563 = 1.689$ sq. in. In addition, there are two rivets whose centers lie 2 inches from the plane, for each of which, according to the rule, there must be deducted three-eighths of the area for one rivet; the deduction for these holes is, then, $2 \times \frac{3}{8} \times .563 = .422$ sq. in. Then, the total deduction is $1.69 + .42 = 2.11$ sq. in., and the net section is

$$6.75 - 2.11 = 4.63 \text{ sq. in. Ans.}$$

If the plane $g g$ is considered instead of $p p$, it can be seen that there are four rivets whose centers are 2 in. from the plane. In such a case, when the rivets are on different sides of the plane, it is customary to consider only those on one side.

58. Countersunk Rivets.—The holes for countersunk rivets are larger than those for rivets with full head. The last line in Table XXVII gives the amount by which the area of cross-section of a rivet hole is increased for a countersunk head for rivets from $\frac{1}{2}$ to 1 inch in diameter. In finding the net section of a member in which there are countersunk heads, these additional areas should be allowed for.

EXAMPLES FOR PRACTICE

1. From Table XXVI, find the radius of gyration with respect to an axis parallel to the adjacent legs and half way between them for: (a) two $4'' \times 3'' \times \frac{1}{2}''$ angles with the short legs $\frac{1}{2}$ inch apart and

parallel; (b) two $6'' \times 4'' \times \frac{3}{4}''$ angles with the short legs $\frac{3}{4}$ inch apart and parallel.

$$\text{Ans. } \begin{cases} (a) \ 2.01 \text{ in.} \\ (b) \ 3.09 \text{ in.} \end{cases}$$

2. The built-up section shown in Fig. 12 is composed of one $20'' \times \frac{1}{2}''$ plate and two 15-inch 40-pound channels arranged as shown. Find: (a) the distance of the center of gravity from the upper flanges of the channels; (b) the moment of inertia with respect to the axis $X'X$; (c) the moment of inertia with respect to the axis $Y'Y$; (d) the radius of gyration with respect to the axis $X'X$; (e) the radius of gyration with respect to $Y'Y$.



FIG. 12

$$\text{Ans. } \begin{cases} (a) \ 5.19 \text{ in.} \\ (b) \ 1,117 \text{ in.}^2 \\ (c) \ 1,433 \text{ in.}^2 \\ (d) \ 5.77 \text{ in.} \\ (e) \ 6.54 \text{ in.} \end{cases}$$

3. The built-up section shown in Fig. 13 is composed of two $16'' \times \frac{5}{8}''$ plates and four $4'' \times 4'' \times \frac{5}{8}''$ angles, arranged as shown. Find: (a) the moment of inertia with respect to the axis $X'X$; (b) the moment of inertia with respect to the axis $Y'Y$; (c) the radius of gyration with respect to the axis $X'X$; (d) the radius of gyration with respect to $Y'Y$.

$$\text{Ans. } \begin{cases} (a) \ 1,331 \text{ in.}^2 \\ (b) \ 1,411 \text{ in.}^2 \\ (c) \ 5.88 \text{ in.} \\ (d) \ 6.06 \text{ in.} \end{cases}$$

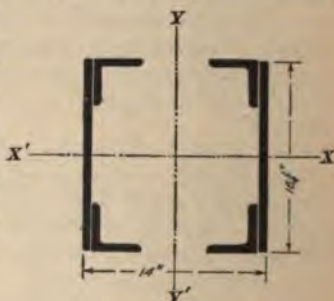


FIG. 13

4. A tension member consisting of two 12-inch 30-pound channels has its sectional area reduced by the

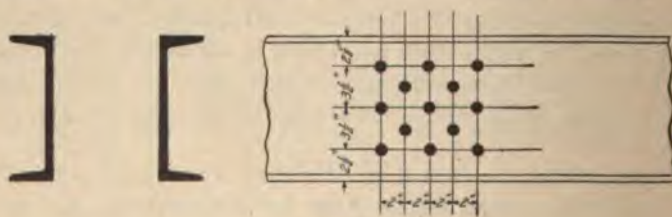


FIG. 14

group of $\frac{3}{4}$ -inch rivets shown in Fig. 14, in each web. Find the net section of the member.

$$\text{Ans. } 14.96 \text{ sq. in.}$$

5. A tension member is composed of two plates $15 \text{ in.} \times \frac{1}{2} \text{ in.}$ and $8 \text{ in.} \times \frac{9}{16} \text{ in.}$, respectively, and four angles $3\frac{1}{2} \text{ in.} \times 3\frac{1}{2} \text{ in.} \times \frac{9}{16} \text{ in.}$,

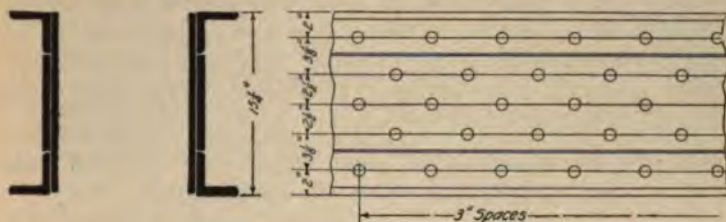


FIG. 15

arranged as shown in Fig. 15. Find the net section if the rivets in each side are $\frac{7}{8}$ inch in diameter and spaced as shown.

Ans. 32.10 sq. in.

PINS, EYEBARS, AND LATERALS

PINS

59. Truss Pins and Nuts.—In pin-connected trusses, a large circular hole is bored at right angles to the truss through each member that connects at a joint, and a pin, somewhat resembling a large bolt, and having a diameter about $\frac{1}{32}$ inch less than that of the hole, is passed through to hold the members in place. The projecting ends of the pin are smaller in diameter than the body of the pin, and a large nut is turned on to each end to keep the members packed together.

The distance between the outside surfaces of the outside members that connect at a joint is called the **grip** of the pin. In order to give the outside members a good bearing, the main body of the pin is made about $\frac{1}{4}$ inch longer at each end than the grip, and the nuts are recessed so that they enclose the projecting ends and bear firmly against the outside members.

Table XXVIII gives the standard dimensions of screw ends and nuts for truss pins varying in diameter from 2 to 8 inches.

60. Lateral Pins.—For the connections of members of lateral and sway systems in pin-connected trusses, a pin

without nuts, often called a **lateral pin**, is sometimes used, although the type of pin described in the preceding article is to be preferred. Lateral pins are turned from rods of slightly larger diameter than the required diameter of the pin, leaving a shoulder at one end; the other end is tapered slightly to facilitate passing the pin through the hole. The tapered end is held in place by a split key called a **cotter pin**. Table XXIX shows the dimensions of heads and cotter pins for the usual sizes of lateral pins.

EYEBARS

61. General Description and Properties.—The principal dimensions of **eyebars**, a special form of tension member used in pin-connected trusses, are given in Table XXX. These bars are first rolled to uniform width and thickness, then the ends are heated and upset to circular forms called **heads**, the thickness remaining the same. Holes are bored in the heads so that they can connect to the pins at the joints. The sizes of heads given in the table were determined from the results of numerous experiments on eyebars, and are those that have proved in actual practice to be sufficient to develop the full strength of the respective bars. In designing tension members for pin-connected trusses, it is, therefore, simply necessary to consider the strength of the body of the bar, as the heads will be strong enough.

The right-hand column of Table XXX gives the additional length U of bar required beyond the center of the pin hole to form one head. The widths given in the first column of the table are standard widths and no others should be used in designing. The thicknesses given in the second column are the minimum thicknesses; any greater thickness can be obtained.

When thicknesses greater than about 2 inches are required, it is customary to use two or more bars side by side. The diameters of pinholes given in the fourth column are the largest holes that can be bored in the heads of the respective bars without decreasing the strength. Any smaller hole may be used.

EXAMPLE.—An eyebar 6 inches wide and 1 inch thick connects to two pins 6 inches in diameter and 18 feet 6 inches center to center. To find the required length of bar before upsetting the ends to form the heads.

SOLUTION.—It will be seen from Table XXX that the first 6-inch bar cannot have a pin larger than $5\frac{1}{4}$ in. in diameter in its heads, so it is necessary to consider the second. This allows a pin $6\frac{1}{4}$ in. in diameter; and, as the pins in the example are 6 in., this size of head may be used. In the right-hand column, the additional length necessary to form one head is given as 2 ft. 3 in.; as there are two heads, the length of the bar before upsetting is

$$18 \text{ ft. 6 in.} + 2 \times (2 \text{ ft. 3 in.}) = 23 \text{ ft. Ans.}$$

62. Adjustable Eyebars.—When there are two diagonals in the same panel, that is, a main diagonal and a counter, it is difficult to manufacture them so that all the holes in their ends will exactly fit the pins. In such cases, it is customary to make the counter adjustable; that is, to manufacture it in such a way that its length can be slightly increased or decreased, if desired. Table XXXI gives the principal dimensions of **adjustable eyebars**; they are used in pairs, one end of each bar having a flat circular head and pinhole, as given in Table XXX, and the other an upset screw end, as given in Table XXXI. The additional length of bar necessary to form an upset screw end is given in Table XXXI; that necessary to form a flat circular head may be taken from Table XXX. Owing to the method of manufacture, it is impossible to get an adjustable eyebar less than about 7 feet in length; for this reason, in designing, it is well to make one end of an adjustable member about 7 feet long.

63. Turnbuckles.—The screw ends of the two adjustable eyebars that form a counter are brought to within 3 inches

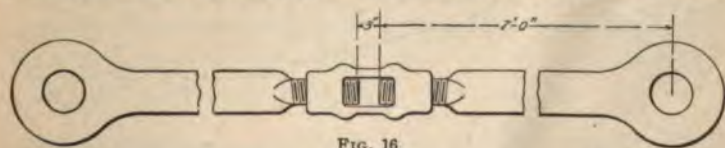


FIG. 16

of each other and connected by means of an appliance called a **turnbuckle** (see Fig. 16). The bars are screwed into the turnbuckle until the member has the desired length. The

threads on the ends of the two bars run in opposite directions, so that, by twisting the turnbuckle, the length of the member may be increased or decreased as desired. Table XXXII gives the principal dimensions of the turnbuckles that are most used in bridge work at the present time. They are called **open turnbuckles**, to distinguish them from less desirable forms, called **closed turnbuckles**, that resemble long nuts. Turnbuckles are made of wrought iron, and are manufactured of such dimensions as will give them greater strength than the bars for which they are designed.

EXAMPLE.—A counter connects to two $3\frac{3}{4}$ -inch pins, 22 feet center to center, and is composed of a pair of adjustable eyebars 4 inches wide and $\frac{3}{4}$ inch thick. What is the length of each part before upsetting?

SOLUTION.—The short end, after upsetting, will be 7 ft. long; and the other end, 22 ft. - 7 ft. 3 in. = 14 ft. 9 in. long. Table XXXI gives the additional length necessary to form a screw end as 1 ft. $\frac{3}{4}$ in.; Table XXX gives the additional length necessary to form a flat circular head as 1 ft. $5\frac{1}{2}$ in. Then, the original length of the short end is

$$7 \text{ ft.} + 1 \text{ ft. } \frac{3}{4} \text{ in.} + 1 \text{ ft. } 5\frac{1}{2} \text{ in.} = 9 \text{ ft. } 6\frac{1}{4} \text{ in.}$$

and that of the long end is

$$14 \text{ ft. } 9 \text{ in.} + 1 \text{ ft. } \frac{3}{4} \text{ in.} + 1 \text{ ft. } 5\frac{1}{2} \text{ in.} = 17 \text{ ft. } 3\frac{1}{4} \text{ in.} \quad \text{Ans.}$$

LOOP-WELDED AND LATERAL RODS

64. Loop-Welded Rods.—Fig. 17 shows another form of member sometimes used for counters and lateral rods, although it is being superseded by the adjustable eyebar. It is called a **loop-welded rod**, and is made from a straight

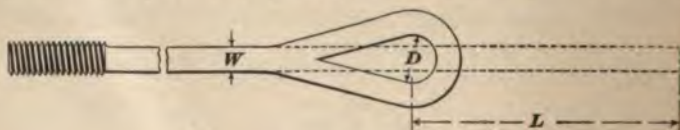


FIG. 17

square rod; one end is upset for a screw end, and the other made in the form of a loop by bending around a circular pin and welding the free end to the main body of the bar. On account of the difficulty and uncertainty in welding steel,

these rods are usually made of wrought iron. They are used in pairs connected by a turnbuckle in the same way as adjustable eyebars. The additional length of rod required to form an upset screw end can be found from Table III; that required to form a loop end is given approximately by the formula

$$L = 3.75 (W + D)$$

in which W = width of bar, in inches;

D = diameter of pin, in inches;

L = additional length beyond center of pin hole, in inches.

EXAMPLE.—A loop-welded counter consisting of a 2-inch square rod connects on two 4-inch pins 21 feet apart. To find the required lengths of straight rod to form the member.

SOLUTION.—One end may be made 7 ft. long, and the other 21 ft. — 7 ft. 3 in., or 13 ft. 9 in. long. Table III gives the additional length to form an upset screw end as $4\frac{3}{4}$ in. By the formula, the additional length to form a loop end is $3.75 \times (4 + 2) = 22.50$ in., or 1 ft. $10\frac{1}{2}$ in. Then, the length of the short piece must be

$$7 \text{ ft.} + 4\frac{3}{4} \text{ in.} + 1 \text{ ft. } 10\frac{1}{2} \text{ in.} = 9 \text{ ft. } 3\frac{1}{4} \text{ in.,}$$

and that of the long piece,

$$13 \text{ ft. } 9 \text{ in.} + 4\frac{3}{4} \text{ in.} + 1 \text{ ft. } 10\frac{1}{2} \text{ in.} = 16 \text{ ft. } \frac{1}{4} \text{ in.} \quad \text{Ans.}$$

65. Lateral Rods.—Table XXXIII gives the principal dimensions of flat circular heads, similar to eyebar heads, for square and round rods, as sometimes used for the connection of members of lateral and sway systems. When so used, the rods are sometimes made in one piece, with a head at each end, and sometimes in two pieces, connected by a turnbuckle. In the latter case, there should be 3 inches between the ends of the pieces for adjustment. As in the case of adjustable eyebars, it is well to make one end shorter than the other, but the short end should be not less than 7 feet in length.

66. Clevises.—Table XXXIV gives the principal dimensions of clevises used for connecting lateral rods to pins to which other lateral rods connect. Instead of forming a head on the end of the rod, an upset screw end is formed, which is screwed into the clevis.

to provide for the tendency of the column to buckle or bend sidewise. In *Strength of Materials*, Part 2, it was explained that a column with fixed ends is stronger than one with the ends hinged or not fixed. The compression members of pin-connected trusses, except the top chord when made continuous, are hinged or free to turn around the pins in their ends. The compression members of riveted trusses are fixed in direction at the ends by being riveted to the gussets or connection plates at the joints, and are, therefore, theoretically stronger than pin-connected columns. When riveted trusses deflect, however, the gussets or connection plates tend to twist, and this twisting produces bending in the members that connect to the gussets, giving rise to what are known as **secondary stresses**, whose magnitudes cannot be computed. On this account, the advantage of riveted columns over pin-connected columns in bridge work is ignored, and the same reduction formula is used to find the allowable working stress in both forms.

71. Various formulas for compression are in use at the present time, all of them being based on the ratio of the unsupported length of the column to the least radius of gyration of its cross-section. The formula now most frequently used in practice has the following general form:

$$s_c = \frac{s}{1 + \frac{l^2}{c r^2}}$$

in which s_c = allowable working stress for column;
 l = unsupported length of column;
 r = least radius of gyration of cross-section;
 c = a constant determined by experiment.

Different specifications give different values for s and c .

In *Bridge Specifications*, s is taken as 16,000 pounds per square inch, and c , as 18,000. The formula then becomes

$$s_c = \frac{16,000}{1 + \frac{l^2}{18,000 r^2}}$$

The values of l and r should be given in the same units; that is, both should be in feet, or both in inches. For

example, if l is 12 feet, or $12 \times 12 = 144$ inches, and r is 3 inches,

$$s_c = \frac{16,000}{1 + \frac{144 \times 144}{18,000 \times 3 \times 3}} = \frac{16,000}{1.128} = 14,180 \text{ lb. per sq. in.}$$

Table XXXV gives the values of s_c for various values of $\frac{l}{r}$.

72. In designing compression members, it is customary to decide on the general form of the cross-section, and then determine the approximate value of the radius of gyration and dimensions of the section, as explained in Art. 47; the length of the member is generally known, and the approximate value of $\frac{l}{r}$ can be computed. The allowable working

stress corresponding to this value of $\frac{l}{r}$ is then found from Table XXXV, and the required area of cross-section found by dividing the total compression by the allowable working stress. With this area as a basis, the cross-section is determined, its radius of gyration and the actual value of $\frac{l}{r}$ are computed, and the working stress corresponding to the corrected value of $\frac{l}{r}$ is found from Table XXXV. The area of cross-section is then revised, if necessary, to make it correspond with the new working stress.

EXAMPLE.—The total compressive stress in a member 20 feet long is 281,000 pounds, and the approximate value of the radius of gyration is 4.8 inches. What is the trial value of the area of cross-section of the member that would be used in designing?

SOLUTION.—The length of the member is 20 ft., or $20 \times 12 = 240$ in. The value of $\frac{l}{r}$ is $\frac{240}{4.8} = 50$. Consulting Table XXXV, the working stress corresponding to a value of 50 for $\frac{l}{r}$ is found to be 14,050 lb. per sq. in. Then, since the total compressive stress is 281,000 lb., the required area of cross-section is

$$281,000 \div 14,050 = 20 \text{ sq. in. Ans.}$$

73. Shearing.—The intensity of shearing stress on any member subjected to a shearing force at right angles to its

length is usually found by dividing the stress by the area of cross-section of the member. In designing, the shearing stress and the allowable working stress are known, and the required area of cross-section is found by dividing the shearing stress by the working stress.

74. In wide thin plates subject to a shearing stress in the plane of the plate, there is a tendency to **buckle**, or bend outwards; and when the intensity of stress is sufficient, this is resisted by stiffening angles a, a, a , Fig. 19, riveted to

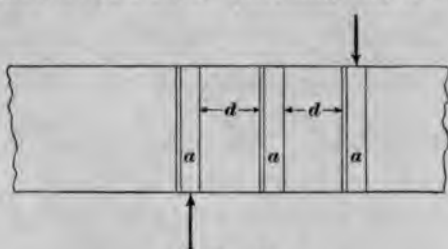


FIG. 19

the plate and parallel to the direction of the shearing stress.

The allowable working stress for shearing in wide plates is found by means of a formula similar to the formula

used in finding the working stress for compression. Different engineers make use of different formulas, the one most used in bridge work at the present time being

$$s_s = \frac{12,000}{1 + \frac{d^2}{3,000t^2}}$$

in which s_s = working stress;

t = thickness of plate;

d = width of plate, or, if plate is stiffened by angles closer together than width of plate, the clear distance between the stiffener angles, as shown in Fig. 19.

For example, if a plate 24 inches wide and $\frac{1}{2}$ inch thick, with no stiffeners or with stiffeners farther apart than 24 inches, is subject to a shearing stress, the allowable working stress for shear is

$$s_s = \frac{12,000}{1 + \frac{24 \times 24}{3,000 \times .5 \times .5}} = 6,790 \text{ lb. per sq. in.}$$

If, however, the stiffeners are closer together than the width of the plate, say 12 inches apart, the allowable working stress for shear is

$$1 + \frac{12,000}{3,000 \times .5 \times .5} = 10,070 \text{ lb. per sq. in.}$$

75. Table XXXVI gives curves that show the allowable working stresses in plates from $\frac{1}{4}$ to 1 inch in thickness, and from 0 to 100 inches in width, calculated from the formula in Art. 74. To find by means of the curve the allowable working stress in the first case considered above, we glance at the left-hand side and find 24, corresponding to an unsupported width of 24 inches; we then glance horizontally across to the curve for a $\frac{1}{2}$ -inch plate, marked $\frac{1}{2}$, and then vertically downwards to the bottom line, on which we find 6,800, which is the allowable working stress, in pounds per square inch.

In the case of plate girders, the flange angles are riveted to the edge of the web, and the unsupported distance is taken equal to the clear distance between the vertical legs of the flange angles, or between stiffeners, whichever is the smaller.

76. Bending.—The allowable working stress in bending on rolled sections is usually the same as that allowed in tension. When the unsupported length of the compression portion is greater than a certain number of times its width, usually twenty times, it is customary to decrease the allowable working stress in compression by means of some reduction formula. A formula for this purpose will be given elsewhere.

BRIDGE MEMBERS AND DETAILS

(PART 2)

DETAILS

RIVETED JOINTS

RIVET VALUES

NOTE.—The tables referred to in this Section are found in *Bridge Tables*.

1. Introductory.—Rivets are used for two principal purposes; namely, to hold the several parts of a built-up member together so as to make them act as one piece, and to transmit stress from one member or part of a member to another. In the first case, the stresses on the rivets are not calculated; the rivets are spaced according to established practical rules that will be given in *Bridge Specifications*. In the second case, the stresses on the rivets and the number of rivets required to transmit given stresses must be computed in order that the allowable stress on a rivet may not be exceeded.

It has been found by experiment that, when the spaces between centers of rivets are less than three times the diameter of the rivet, and the rivets are not nearer the edge of a plate than one and one-half times the diameter, the only conditions of strength that must be considered are the resistance offered by the rivet to shearing on one or more

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sections between the members connected, and the resistance to crushing offered by the plates bearing on the rivets.

2. Friction of Riveted Joints.—As explained in *Bridge Members and Details*, Part 1, rivets are driven hot, and in cooling contract and hold firmly together the parts through which they pass. This causes a certain amount of friction between the parts, which helps to transmit stress from one member or part of a member to another. It is impossible to determine with any certainty how great this friction is, and it is customary to ignore it in the design of riveted joints, the shearing and bearing resistances alone being considered.

3. Shearing Value.—The maximum shearing stress that is allowable on a rivet is called the **shearing value** of

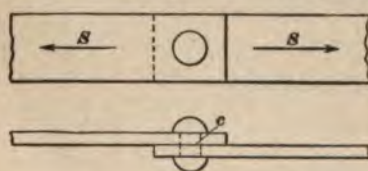


FIG. 1

the rivet, or the **value** of the rivet, in shear. If a rivet connects two plates, as represented in Fig. 1, the stress in one plate is transmitted to the other by means of the rivet, and the area

subjected to shear is the area of cross-section of the rivet. As there is but one section c of the rivet subjected to shear,

the rivet is said to be in **single shear**. When a rivet connects two members, as represented in Fig. 2, the rivet is subjected to shear at two sections d and e , and is said to be in **double shear**.

In calculating the area of cross-section of a rivet, the nominal diameter is used.

The shearing value of a rivet is found as follows:

Rule.—To find the value of a rivet in single shear, multiply the area of cross-section of the rivet by the allowable intensity of stress in shear; to find the value of a rivet in double shear, multiply twice the area of cross-section of the rivet by the allowable intensity of stress in shear.

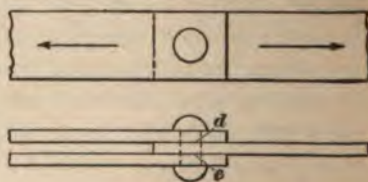


FIG. 2

EXAMPLE.—What is the value of a $\frac{7}{8}$ -inch rivet (*a*) in single shear, and (*b*) in double shear, if the working stress is 11,000 pounds per square inch?

SOLUTION.—(*a*) Consulting Table I, the area of cross-section of a $\frac{7}{8}$ -inch round rod is found to be .601 sq. in. Since the working stress is 11,000 lb. per sq. in., the value in single shear is

$$.601 \times 11,000 = 6,610 \text{ lb. Ans.}$$

(*b*) The value in double shear is

$$2 \times .601 \times 11,000 = 13,220 \text{ lb. Ans.}$$

4. **Bearing Value.**—The resistance to crushing offered by the members that bear on a rivet depends on the thickness of the plates that transmit the stress to and from the rivet, and on the diameter of the rivet. When a rivet bears on a plate, it is in contact with the inside of the rivet hole, as at *abc*, Fig. 3, the intensity of bearing or pressure being greater at *b* than at *a* and *c*; and it is difficult to determine the exact area of bearing. In practice, it is customary to assume that the area of bearing is equal to the product

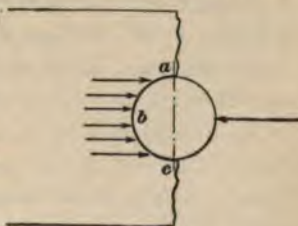


FIG. 3

of the nominal diameter of the rivet and the thickness of the plate. The working strength of the plate is commonly called the **bearing value** of the plate, and is equal to the product of the diameter of the rivet, thickness of the plate, and allowable intensity of stress.

The bearing value of a plate on a rivet is frequently spoken of as the bearing value of the rivet on the plate, it being understood that the bearing value of the plate is meant.

EXAMPLE.—If the working stress in bearing is 22,000 pounds per square inch, what is the bearing value of a plate $\frac{1}{2}$ inch thick on a rivet $\frac{7}{8}$ inch in diameter?

SOLUTION.—The bearing value is equal to the continued product of the diameter, $\frac{7}{8}$ in., the thickness of the plate, $\frac{1}{2}$ in., and the working stress, 22,000 lb. per sq. in.; that is, the bearing value is

$$\frac{7}{8} \times \frac{1}{2} \times 22,000 = 9,625 \text{ lb. Ans.}$$

5. **Tables of Rivet Values.**—Tables XXXVII to XL give the values of rivets from $\frac{1}{2}$ to 1 inch in diameter, in

single and double shear, for working stresses of from 6,000 to 11,000 pounds per square inch in shear, and the bearing values of plates $\frac{1}{4}$ to $\frac{1}{2}$ inch in thickness on rivets $\frac{1}{2}$ to 1 inch in diameter, for working stresses from 12,000 to 22,000 pounds per square inch in bearing. For example, if it is desired to know the value in double shear of a rivet $\frac{3}{4}$ inch in diameter for an allowable stress in shear of 9,000 pounds per square inch, Table XXXIX is consulted. In the fourth column from the left, headed Shear Values at 9,000 Pounds per Square Inch, Double Shear, is found, opposite $\frac{3}{4}$ in the first column, the value 7,950 pounds, which is the required value. In like manner, if it is desired to know the bearing value of a $\frac{9}{16}$ -inch plate on a $\frac{7}{8}$ -inch rivet, for an allowable stress in bearing of 15,000 pounds per square inch, Table XXXVIII is consulted. In the column headed $\frac{9}{16}$, and opposite $\frac{7}{8}$ in the first column, the required value, 7,380 pounds, is found.

In Tables XXXVII to XL, the bearing values below and to the left of the dotted lines are less than the values in single shear; those below and to the left of the heavy full lines are less than the values in double shear; and those above and to the right of the heavy full lines are greater than the values in double shear.

6. Critical Value of a Rivet.—Each rivet may be said to have three values; namely, its shearing value, and the bearing values of the members from which and to which the stress is transmitted. For example, in Fig. 4, the stress is transmitted from the two angles a, a to the plate b . The rivets bear on the two angles, are in double shear, and bear on the plate. If the rivets are $\frac{7}{8}$ inch in diameter, the angles $\frac{3}{8}$ inch thick, the plate $\frac{1}{2}$ inch thick, and the working stresses in shear and bearing are 9,000 and 18,000 pounds per square inch, respectively, Table XXXIX shows that the value in double shear is 10,820 pounds; in bearing on the $\frac{1}{2}$ -inch plate, 7,880 pounds; and in bearing on two $\frac{3}{8}$ -inch angles ($\frac{3}{4}$ inch total thickness), 11,810 pounds. The smallest of these three values—in this case the bearing on the $\frac{1}{2}$ -inch plate—is called the **critical value** of the rivet, and sometimes simply *the* value of the rivet.

7. In making a drawing of a riveted member, it is customary to show all the rivets or to indicate their location in some way. When several views of the member are drawn, the rivets are usually represented by the conventional signs explained in *Bridge Members and Details*, Part 1. When only one view is shown, and frequently also when more views than one are shown, the rivets are represented in other ways. For example, in Figs. 1, 2, and 4, the rivets are represented

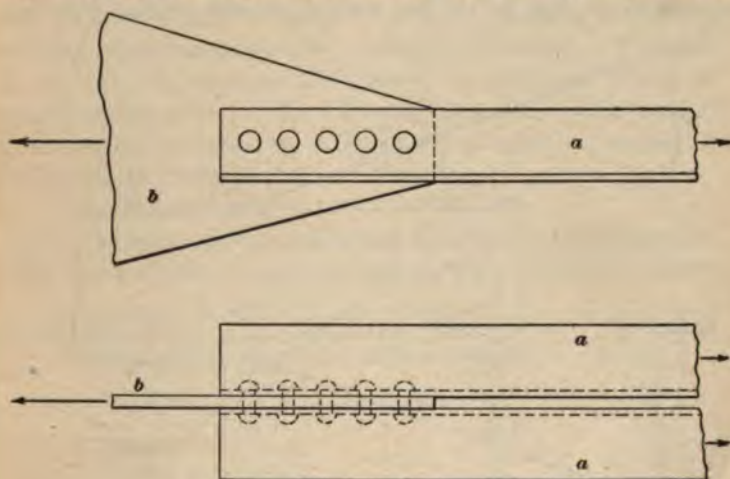


FIG. 4

in the upper part of each figure by circles; and in the lower portion they are represented in elevation. In Fig. 5, they are shown in but one view. In Fig. 6, the rivets are shown only in (a), but in that view some are shown in plan and some in side elevation. In Fig. 7, the conventional signs are not shown, but the location of each rivet is represented by a small cross; this method is frequently used when there are many rivets to be drawn.

8. **Required Number of Rivets.**—The number of rivets required to connect two members is found by dividing the stress that is to be transmitted, by the critical value of one rivet. For example, if the stress that is

transmitted from the angles to the plate in Fig. 4 is 38,000 pounds, and other conditions are as given in Art. 6, then, since the critical value of a rivet has been found to be 7,880 pounds, the required number of rivets is $38,000 \div 7,880 = 4.82$. The next whole number, 5 in this case, is always provided.

EXAMPLE.—A 10-inch 20-pound channel, in which the stress is 82,500 pounds, is connected to a $\frac{7}{8}$ -inch plate by $\frac{7}{8}$ -inch rivets, as represented in Fig. 5. If the working stresses on the rivet are

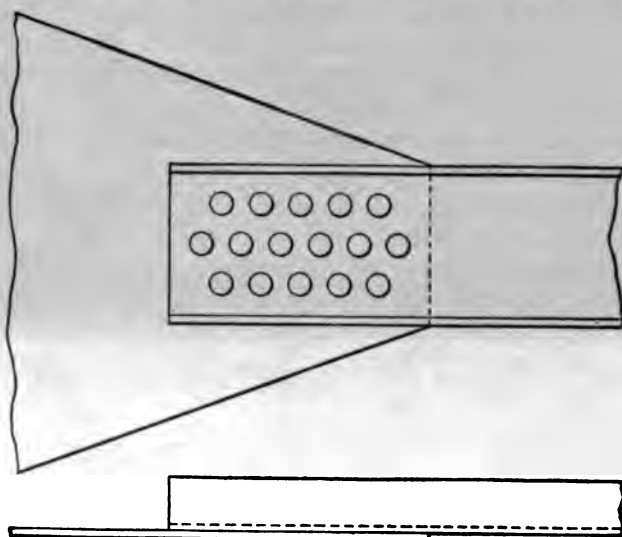


FIG. 5

9,000 pounds per square inch in shearing and 18,000 pounds per square inch in bearing, how many rivets are required to transmit the stress from the channel to the plate?

SOLUTION.—Consulting Table XIII, the thickness of the web of a 10-in. 20-lb. channel is found to be $\frac{3}{8}$ -in. Consulting Table XXXIX, the value of a $\frac{7}{8}$ -in. rivet in bearing on a $\frac{7}{8}$ -in. plate is found to be 6,890 lb.; in single shear, 5,410 lb.; and in bearing on the $\frac{3}{8}$ -in. web of the channel, 5,910 lb. As the value 5,410 lb. of a $\frac{7}{8}$ -in. rivet in single shear is the smallest, it is the critical value of the rivet. Then, since the total stress is 82,500 lb., the required number of rivets is

$$82,500 \div 5,410 = 15.25, \text{ or, say, } 16. \text{ Ans.}$$

SPICES IN BUILT-UP MEMBERS

9. Length of Members.—Bridge members composed of built-up shapes are occasionally made as long as 80 feet. These members are exceedingly difficult to handle, both on account of their weight and on account of their size. Their weight, too, is likely to cause excessive stresses while they are being handled. In general, it is advisable to manufacture long members, such as chords of trusses, in several sections, each being from 30 to 60 feet in length. The ends of the several sections are planed so that they will bear against one another, and additional plates and angles, called **splice plates** and **splice angles**, are riveted to the ends of the members where they come together.

10. Forms of Splices.—Fig. 6 shows the elevation (a) and cross-section (b) of a spliced member composed of two

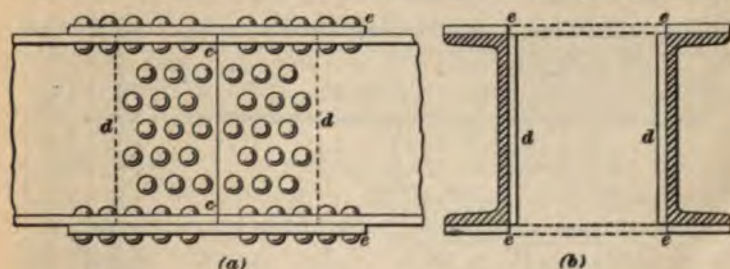


FIG. 6

channels. The ends of the two sections are brought together and placed in contact at *c, c*. The plates *d, d* are riveted to the inside of the channels, and the plates *e, e* to the flanges of the channels. Both *d* and *e* are splice plates. In some cases, the plates *e* are made the full width of the member, as shown by dotted lines. The area of cross-section of the splice plates must in every case, both for tension and for compression members, equal the area of cross-section of the member. Some engineers depend on the bearing area at *c, c* between the ends of the two portions to transmit compression, and simply place enough splice plates and rivets at

the joint to hold the member in line. The best practice at the present time, however, is to make the area of the splice plates equal to the area of the member, and to place enough rivets in each splice plate to transmit the amount of stress that is transmitted by the plate. Further details regarding splices will be given in subsequent Sections.

JOINTS IN RIVETED TRUSSES

11. Typical Joint.—Fig. 7 represents the top view (*a*), side view (*b*), and end view (*c*) of a typical top chord joint in a riveted truss. In parallel-chord trusses, the top chord *de*

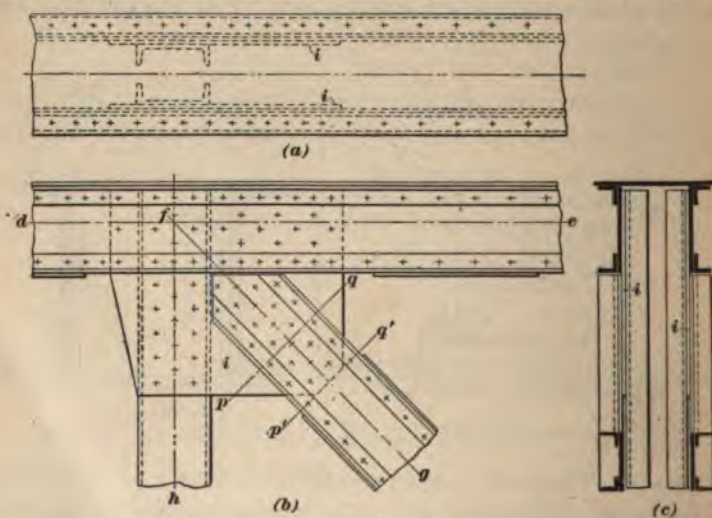


FIG. 7

is made continuous; that is, the angles and plates composing it are not joined at the panel points, unless it is desired to splice it at such places. The line *dfe* near the center of gravity of the chord is called the center line of the member, and the lines *fg* and *fh*, through the centers of gravity of the web members, intersect the line *dfe* and each other at the panel point *f*. The plates *i*, called **gussets**, or **web connection plates**, are riveted to the insides of the chord

member, and the web members are riveted to the inside or outside of these plates; in Fig. 7, fg is shown riveted to the outside and fh to the inside. The location of the web members with respect to the gusset plates depends on other details. Other joints will be described in subsequent Sections.

12. Rivets in Connection Plates.—It is customary to assume that one-half the stress in each web member is transmitted to each gusset. The number of rivets required to connect each side of a web member to a gusset is found by dividing one-half the stress in the web member by the value of one rivet. The stress transmitted to the chord by the connection plate is equal to the algebraic sum of the Λ components of the stresses in the web members that connect at the joint. It is seldom necessary to actually compute the number of rivets required to connect the gussets to the chords. If the gussets are made large enough to contain sufficient rivets to transmit the stresses to and from the web members, and are riveted to the chords sufficiently to hold them firmly together, say with a rivet pitch of 3 inches, there will invariably be enough rivets.

13. Size of Gussets.—It is impossible to calculate accurately the stresses in the gussets, and, therefore, to determine their required dimensions with exactness. It is well to see, however, that the area of cross-section on such a section as pq , Fig. 7, at right angles to the diagonal, is sufficient to resist the stress transmitted to the gusset at that point. The amount of stress transmitted to the gusset at such a section can be found by multiplying the value of one connecting rivet by the number of rivets in the gusset between the section considered and the edge $p'q'$ of the plate. It is always advisable to have an excess of material in the gusset at the section considered. In practice, gussets should never be made less than $\frac{7}{16}$ inch thick for highway-bridge trusses nor less than $\frac{1}{2}$ inch for railroad-bridge trusses.

14. Clearance.—When a member is riveted to the inside of the gussets, it is customary to make it $\frac{1}{8}$ inch

narrower than the clear distance between the gussets. The object of this is to make it easier to push the member between the gussets when the parts are being put together.

PIN JOINTS

15. Typical Pin Joint.—Fig. 8 represents the side elevation (*a*), end view (*b*), and top view (*c*) of a typical bottom chord joint in a pin-connected truss. The bottom

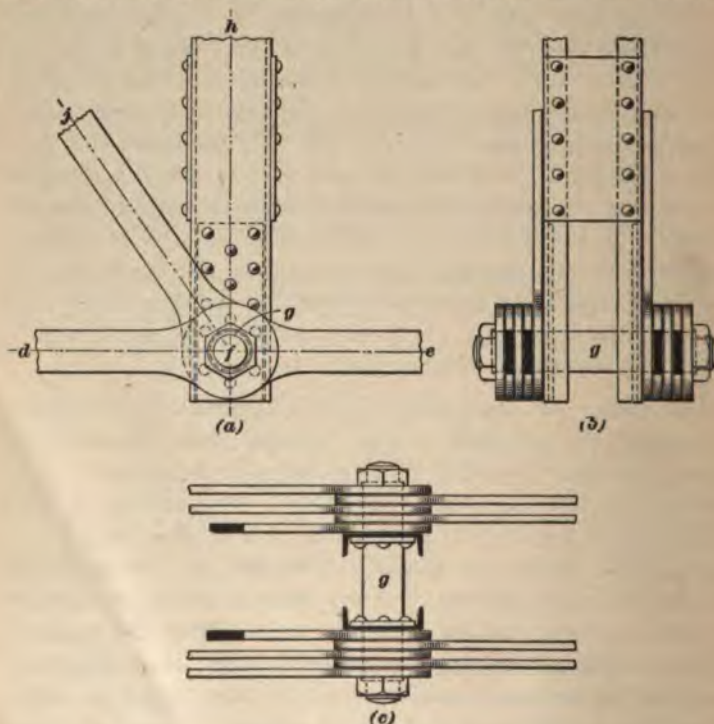


FIG. 8

chord *dfe* and diagonal *fj* are made of eyebars; the vertical *fh* is usually composed of two channels. The pin *g*, which passes through the ends of the members, holds them in position and transmits the stresses from one member to

another. The different members that connect to a pin are placed as close together as possible. It is customary to locate the vertical member nearest the center of the pin, then the diagonals, and then the eyebars in the bottom chord, alternating them as shown. The required size of the pin depends on the stresses to be transmitted and on the relative location of the members. Two conditions are considered in the design of a pin; namely, the intensity of bearing on the members, and the bending moment on the pin.

16. Intensity and Required Thickness of Bearing.

The area of bearing of a member on a pin is found, in the same way as the bearing area of a rivet, by multiplying the diameter d of the pin by the thickness t or width of the bearing. The intensity of pressure s_b is then found by dividing the total stress S in the member by the bearing area. Eye-bars are so designed by the manufacturers that the intensity of bearing need not be considered, if the conditions given in Table XXX are adhered to. In built-up members, however, it is always necessary to compute the intensity of bearing by means of the formula

$$s_b = \frac{S}{td}$$

When the diameter of the pin and the working stress in bearing are known, the required thickness of bearing can be found by the formula

$$t = \frac{S}{s_b d}$$

Table XLI gives, in columns 3, 4, and 5, the values of $s_b d$ for all the ordinary sizes of pins and for customary working stresses. In case a member has not the required thickness, the portion in contact with the pin is reinforced by pin plates.

17. Pin Plates.—Pin plates are placed on the members as represented in Fig. 9, in which c, c are the pin plates. It is customary to assume that the stress in a member is evenly distributed over the thickness or width of bearing; so the amount transmitted by the pin plates can be found by multiplying the total stress by the ratio of the thickness of the

pin plates to the total thickness of bearing. The pin plates must be connected to the member by sufficient rivets to transmit this portion of the stress to the member.

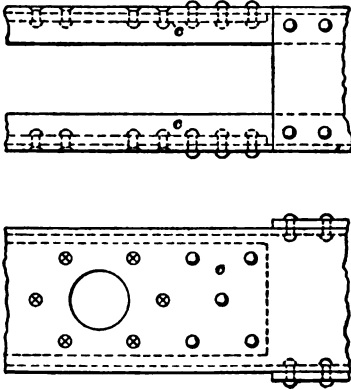


FIG. 9

EXAMPLE.—The total stress transmitted by a member is 220,000 pounds. The thickness of the member in bearing on a 5-inch pin is $\frac{3}{4}$ inch, and the allowable intensity of bearing is 22,000 pounds per square inch. (a) What is the required thickness of bearing? (b) What is the required thickness of pin plates? (c) What is the amount of stress transmitted by the pin plates? (d) If the

rivets connecting the pin plates to the member have a value of 4,860 pounds, how many rivets are required?

SOLUTION.—(a) The required thickness of bearing is found by the formula

$$t = \frac{S}{s_b d}$$

In the present case, $S = 220,000$ lb. Consulting Table XLI, it is seen that $s_b d = 110,000$ lb. Then,

$$t = 220,000 \div 110,000 = 2 \text{ in. Ans.}$$

(b) Since the required thickness is 2 in., and the thickness of the member is $\frac{3}{4}$ in., the required thickness of pin plates is

$$2 - \frac{3}{4} = 1\frac{1}{4} \text{ in. Ans.}$$

(c) Since the required thickness of pin plates is $1\frac{1}{4}$ in., and the total thickness is 2 in., the amount of stress transmitted by the pin plates is

$$\frac{1.25}{2} \times 220,000 = 137,500 \text{ lb. Ans.}$$

(d) Since the amount of stress transmitted by the pin plates is 137,500 lb., and the value of one rivet is 4,860 lb., the number of rivets required to connect the pin plates to the member is

$$137,500 \div 4,860 = 28.3, \text{ or, say, } 29. \text{ Ans.}$$

18. Bending Moment on the Pin.—The required diameter d of the pin is found from the maximum bending moment M on the pin, by means of the formula

$$d = \sqrt[3]{\frac{32 M}{\pi s_f}} \quad (1)$$

in which s_f is the working stress in bending.

As this formula is not convenient for rapid work, it is customary in practice to use a table that gives values of M corresponding to the usual working stresses and diameters of pins. Table XLI gives in columns 6 to 10 the values of the allowable bending moments on different pins for several working stresses. Those bending moments have been computed by means of the formula

$$M = \frac{\pi s_f d^3}{32} \quad (2)$$

For example, if it is known that the bending moment on a pin is 270,000 inch-pounds, and the working stress in bending is 22,000 pounds per square inch, it can be seen by a glance at column 9 that a pin 5 inches in diameter is required. The method of calculating the maximum bending moments on the pins will be given in a subsequent Section.

EXAMPLES FOR PRACTICE

1. If the working stress in shear is 6,000 pounds per square inch, what is the value of a $\frac{3}{4}$ -inch rivet: (a) in single shear? (b) in double shear?

$$\text{Ans. } \begin{cases} (a) & 2,650 \text{ lb.} \\ (b) & 5,300 \text{ lb.} \end{cases}$$

2. If the working stress in bearing is 18,000 pounds per square inch, what is the bearing value of a $\frac{5}{8}$ -inch rivet on a plate: (a) $\frac{5}{16}$ inch in thickness? (b) $\frac{5}{8}$ inch in thickness?

$$\text{Ans. } \begin{cases} (a) & 3,520 \text{ lb.} \\ (b) & 7,030 \text{ lb.} \end{cases}$$

3. A 9-inch 15-pound channel in which the stress is 39,800 pounds is connected to a $\frac{1}{2}$ -inch plate by $\frac{3}{4}$ -inch rivets. If the working stresses are 9,000 pounds per square inch in shear, and 18,000 pounds per square inch in bearing, how many rivets are required?

$$\text{Ans. } 11 \text{ rivets}$$

4. The total stress transmitted by a member is 180,000 pounds. The thickness of the member in bearing on a 4-inch pin is 1 inch, and the allowable intensity of bearing is 18,000 pounds per square inch. (a) What is the required thickness of bearing? (b) What is the required thickness of pin plates? (c) What amount of stress is transmitted by the pin plates?

$$\text{Ans. } \begin{cases} (a) & 2.5 \text{ in.} \\ (b) & 1.5 \text{ in.} \\ (c) & 108,000 \text{ lb.} \end{cases}$$

5. The maximum bending moment on a pin is 157,100 inch-pounds, and the working stress in bending is 25,000 pounds per square inch. What is the required diameter of pin?

$$\text{Ans. } 4 \text{ in.}$$

FLOOR SYSTEMS

FLOORS IN HIGHWAY BRIDGES

19. **Kinds of Wearing Surface.**—The top of the floor, commonly called the **wearing surface**, is, in highway bridges, usually composed of one or two layers of wooden plank supported by wooden or steel beams, called **stringers**,

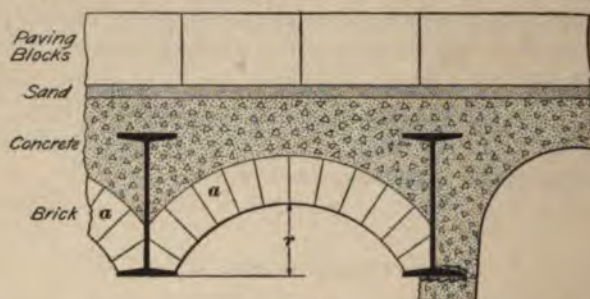


FIG. 10

parallel to the girders or trusses and having a length equal to the panel length of the bridge. When a more durable floor is required, it is made of wooden or granite blocks, or



FIG. 11

of asphalt; the wearing surface is then supported by brick or concrete arches between the stringers, as shown in Fig. 10; by reinforced-concrete slabs, as shown in Fig. 11;

or by buckle plates, as shown in Fig. 12. With each of these types of floors, concrete is brought up to the subgrade of the

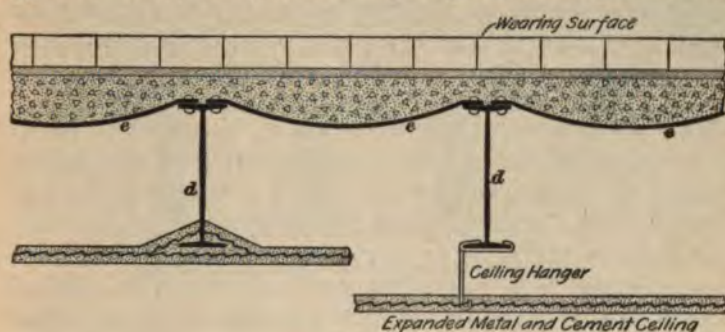


FIG. 12

roadway, as shown, and the blocks or asphalt are placed on top with a layer of sand about 1 inch thick for a cushion coat.

20. Arches Between Beams.—When brick arches are used between the beams, as shown in Fig. 10, one ring of brick *aa* is put in, and concrete is filled in on top. The rise *r* of the arch depends wholly on the depth of stringer, being made greater for a deep than for a shallow stringer. The top of the bricks composing the arch should, in general, be no higher than the top of the stringers. The concrete should be at least 3 inches thick above the stringers. The stringers are spaced about 3 feet apart in this type of floor. There is no necessity to calculate the strength of the arch just described, as it will have ample strength to carry any loads that pass over highway bridges if the stringers are spaced about 3 feet apart.

In the most recent floors of this type, the ring of brick is omitted and the arch is formed of concrete. The crown of the arch is usually kept below the top of the stringers and the concrete at the crown is made not less than about 6 inches in thickness. This construction is shown at the right of Fig. 10. When this type of arch is used, it is customary to continue the concrete below and around the bottom flanges of the stringers for protection against

corrosion. For this purpose, expanded metal is bent so as to have it close to the top and bottom surfaces of the bottom flange, and the concrete is then put in to a thickness of not less than about 2 inches. This is a very serviceable type of floor, and is preferable in many ways to the brick arches.

21. Reinforced-Concrete Slab.—When the type of floor shown in Fig. 11 is used, the stringers are spaced about 3 feet apart, and the concrete slab is made from 6 to 9 inches thick. The amount of reinforcement depends on the load that the bridge is to carry. Ordinarily, not less than about 1 square inch cross-section of steel should be allowed to each foot of length of the slab measured at right angles to the steel rods. In this type of floor, it is customary to extend the concrete down around the stringers so as to protect the steel from corrosion. The method of doing this is shown in Fig. 11.

When reinforced concrete is used for floor slabs, great care should be taken in mixing and placing the concrete as well as in the design of the floor. Poor design and workmanship have been the cause of so many failures in reinforced-concrete construction in the last few years that it is evident that too much care cannot be taken.

22. Buckle Plates.—Fig. 12 shows the type of floor most frequently used when a solid floor is required. The stringers d, d support a thin plate c , on top of which concrete is placed; the wearing surface is placed on a sand cushion resting on the concrete. The plate c is originally flat, and square or very nearly square sections f, f, f, f , Fig. 13, are pressed outwards 2 or 3 inches, forming what are known as **buckles**. This plate can be had in sections about 3 feet wide and containing anywhere from one to eight buckles. The edges g, g of the plate, and short distances h, h between the buckles are made straight and flat so that they can be riveted to the supporting stringers. It has been found that these plates are much stronger when the edges are riveted down than when they are simply supported at the edges, and that they are much stronger when the buckle is turned

downwards, as shown, than when turned upwards in the form of an arch. The standard sizes of buckle plates vary from 2 ft. \times 2 ft. 6 in. by intervals of about 3 inches to 4 ft. \times 4 ft. 6 in. As a rule, it is best to have the buckles

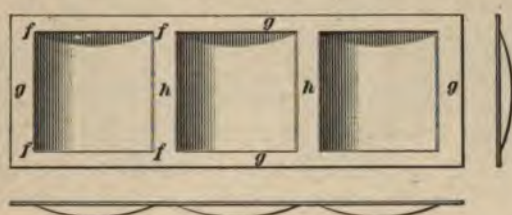


FIG. 13

square or nearly square, and the distances between them about 4 inches; the edges should also be made about 4 inches wide.

Very little is known about the actual strength of buckle plates, but it is known that, if the stringers are spaced not more than 3 feet 6 inches apart, a thickness of $\frac{5}{16}$ inch is sufficient under the sidewalk and $\frac{3}{8}$ inch under the roadway to support any loads that will ever come on them in an ordinary highway bridge. The buckle plates should always be riveted at the edges to the stringers that support them, with $\frac{3}{4}$ -inch rivets spaced about 5 inches apart.

When buckle plates are used in the floors of bridges that cross steam railroads, many pockets are formed by the stringers and floorbeams that extend down below the buckle plates. These pockets become filled with the gases that escape from the smokestacks of locomotives passing under them, and the gases rapidly corrode the steel. On account of trains passing under the bridge, it is extremely difficult to get at these corroded surfaces to clean and paint them. For this reason, it is the best practice, at the present time, to paint carefully and thoroughly all the exposed surfaces of steel under the floor as soon as the bridge is built, and then to construct a ceiling under the bottom flanges of the stringers and floorbeams. The ceiling is usually constructed of expanded metal and concrete 2 or 3 inches in thickness, and so arranged that no pockets are formed. This is

accomplished, in some cases, by supporting the ceiling directly on the bottom flanges of the stringers, as shown at the left of Fig. 12, and in others by placing the ceiling level with the bottom of the floorbeam and supporting the expanded metal by some form of ceiling hanger as shown at the right of Fig. 12.

FLOORS IN RAILROAD BRIDGES

23. Open Floors.—The standard open floor for railroad bridges consists of cross-ties resting on stringers, as shown in Fig. 14. The ties are usually 8 in. \times 8 in. in cross-section, 10 feet long, and spaced about 12 inches center to center. The main rails *a, a* are spiked to the tops of the ties; between

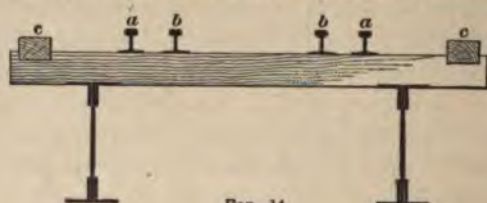


FIG. 14

the main rails on which the cars run, two lines of rails *b, b* are spiked to the ties to act as guard-rails, their duty being to keep derailed cars from moving sidewise. Near the ends of the ties, guard timbers *c, c* are bolted to the tops of the ties to keep derailed cars from going over the side of the bridge and also to keep the ties properly spaced. The floor shown in Fig. 14 is the most economical type of floor, but it is not adapted to all conditions.

It is frequently necessary to adopt a special form of floor that occupies less room below the rail; besides, there are many cases in which it is objectionable to leave open the spaces between the ties. Various forms of solid floors are used to meet special conditions, and some of them will be considered here.

24. Trough Floors.—A style of trough floor used to a considerable extent in the past is shown in cross-section in Fig. 15. The trough *t, t* consists of an upper and a lower

half with inclined flanges riveted together as shown at *a, a*. The trough section is continued straight across from girder to girder or truss to truss, and the ends rest on shelf angles

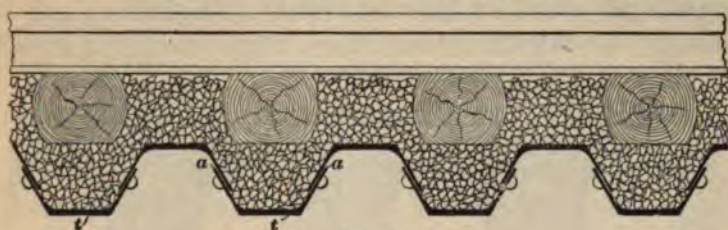


FIG. 15

or on the bottom flanges of the beams, as shown in Fig. 16. The top of the trough is covered with gravel or broken-stone ballast, and the ties are set directly on the ballast in the same way as on solid ground. This form of trough is being superseded by that shown in Fig. 17, which is stronger and can be connected much better to the girders or trusses that support it; in this type, the troughs are rectangular; they are composed of plates and angles, and can be given any desired strength. The ballast and ties occupy the same relative positions with respect to the trough as in Fig. 15. The top of the trough should not be allowed to come closer to the rail than about 2 inches.

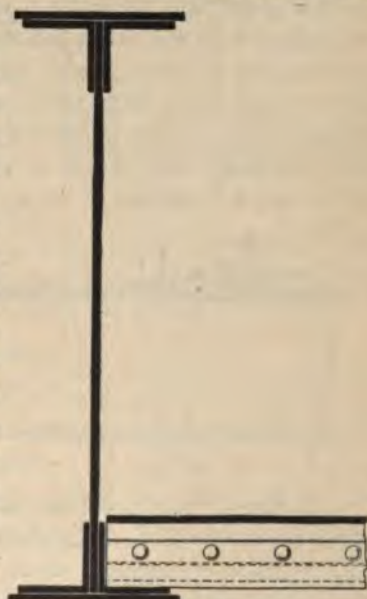


FIG. 16

25. Solid Plate Floors.

Another form of solid floor is made by placing a flat plate *aa*, $\frac{7}{16}$ or $\frac{1}{2}$ inch thick, on top of the beams or stringers, as shown in Fig. 18, and doing away with wooden ties entirely. The rail is placed directly

on top of the plate, and is held in place by small clips *c, c* that are bolted to the top plate. In this type of floor, angles are riveted to the top of the plate *a a*; they are called **guard angles** and serve the same purpose as guard-rails.

Up to the present time, none of the types of solid floors used in railroad bridges have given entire satisfaction. This

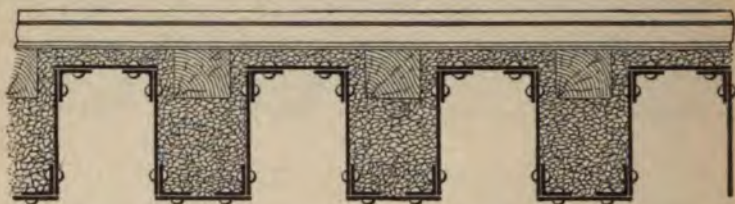


FIG. 17

is due principally to the fact that water and dirt accumulate in contact with the steel and the latter corrodes very rapidly. During the last few years, various forms of reinforced-concrete floors have been tried. These have almost all been modifications of the general type shown in Fig. 11 for highway bridges. The ballast is usually placed on top of the concrete, and a thickness of several inches placed under the ties.

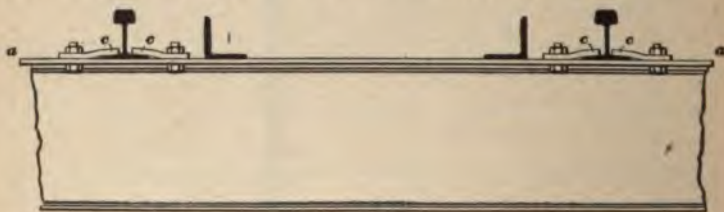


FIG. 18

These floors have not been in use for a sufficient length of time to establish their suitability for this purpose. Some engineers have adopted them as standard for certain conditions; others will not use them on account of the liability of the concrete to crack, owing to the impact and vibration of trains and the deflection of the bridge.

STRINGERS AND STRINGER CONNECTIONS

26. Stringers.—Stringers of floor systems are usually either wooden beams, I beams, or plate girders. When plate girders are used, each flange is generally composed of two angles with no flange plates, although in very long spans flange plates are sometimes used. The design of stringers is no more difficult than that of simple beams, but the design of the connections should be made with care.

27. Stringer Connection Angles.—For the purpose of connecting the stringer to the floorbeam, connection angles are riveted to the ends of the stringers. Fig. 19 represents the side view (*a*) and end view (*b*) of the end of an I-beam stringer furnished with hitch, or connection, angles *c*. The hitch angles are made as long as possible, that is, as long as the straight portion of the web, and riveted tight to the web.

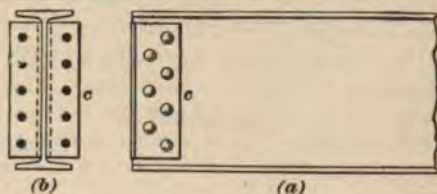


FIG. 19

Fig. 20 represents the side view (*a*) and end view (*b*) of the end of a plate-girder stringer furnished with hitch or connection angles *c*.

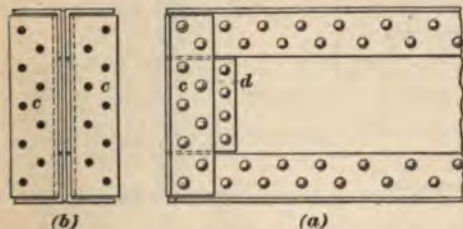


FIG. 20

The hitch angles are made as long as possible, that is, very nearly as long as the clear distance between outstanding legs of the flange angles, and are riveted to the outside surfaces of the flange angles, the space between them and the web between the flange angles being filled by means of fillers *d* having the same thickness as the flange angles. These fillers are usually of greater width

than the connection angle, and are made tight by riveting them to the web outside of the leg of the connection angle, forming what are known as **tight fillers**.

The number of rivets required to connect the connection angles to the end of a stringer is found by dividing the maximum reaction on the stringer by the value of one rivet. For an I-beam stringer, the critical value of the rivet is usually its value in bearing on the web. For a plate-girder stringer with tight fillers, the combined thickness of web and tight fillers is counted as bearing on the rivet, and the critical value of the rivet is usually its value in double shear.

EXAMPLE.—The maximum reaction at the end of the I beam represented in Fig. 19 is 45,000 pounds. How many $\frac{3}{4}$ -inch rivets are required to connect the hitch angles to the web, which is $\frac{1}{2}$ inch in thickness, if the working stresses in shear and bearing are 9,000 and 18,000 pounds per square inch, respectively?

SOLUTION.—Consulting Table XXXIX, the value of a $\frac{3}{4}$ -inch rivet in double shear is found to be 7,950 lb., and in bearing on a $\frac{1}{2}$ -inch web, 6,750 lb. Since the latter is the smaller, it is the critical value of the rivet, and the required number of rivets is

$$45,000 \div 6,750 = 6.7, \text{ or, say, } 7. \text{ Ans.}$$

28. Stringer Connection to Floorbeams.—There are several methods of connecting stringers to floorbeams. The

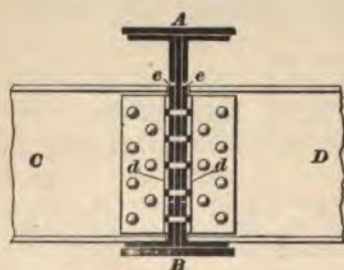


FIG. 21

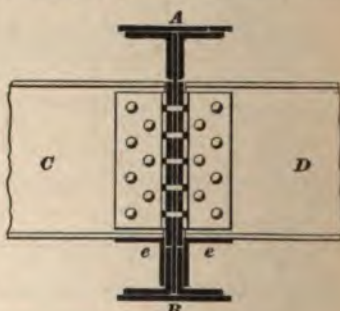


FIG. 22

principal methods are represented in Figs. 21 to 24, in all of which *AB* is the cross-section of the floorbeam, and *CD* are portions of the stringers that connect to them. Fig. 21 shows the connection that is employed when it is desirable to use

stringers and it is necessary to keep the depths of floor as small as possible. The bottoms of the stringers rest directly on top of the bottom flange of the floorbeam. The outstanding legs *d* of the connection angles come in contact with the vertical legs of the bottom flange angles of the floorbeams, and fillers *e* having the same thickness as those angles are placed between the connection angles and the web. This connection can be used only when the depth of the floorbeam is but little greater than that of the stringer.

Figs. 22 and 23 show connections that are sometimes used when the floorbeam is considerably deeper than the stringer

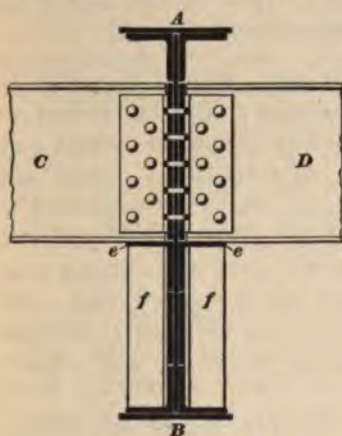


FIG. 23

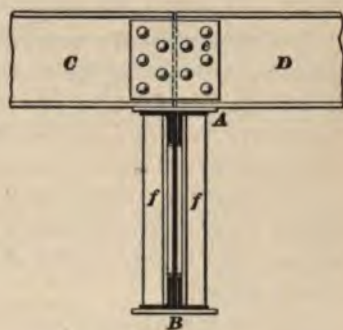


FIG. 24

and the bottom of the stringer is a sufficient distance above the bottom of the floorbeam. A chair or shelf is provided by riveting to the bottom flange or web of the floorbeam a short shelf angle *e* parallel to the flange; and when this is above the bottom flange, as in Fig. 23, inserting short stiffeners *f* between the outstanding legs of the shelf and the bottom flange angles. The connection angles are riveted to the floorbeam web the same as if the chair or shelf were not provided; the latter simply affords additional strength and better distributes the load from the stringer over the floorbeam. The connection shown in Fig. 23 is by far the

best type of connection between stringers and floorbeam, and should be used wherever conditions permit. The designer should endeavor so to arrange his connections that this form can be used.

In deck truss bridges and in through bridges in which the allowable depth of floor is comparatively great, the stringers are sometimes placed directly on top of the top flanges of the floorbeams, as represented in Fig. 24. When they are placed in this position, the ends of the stringers are usually connected by plates e that serve to keep them in line. Stiffeners f having a length of at least one-half the depth of floorbeam and preferably the full depth of the floorbeam should be placed under each stringer.

29. It is very important that there should be sufficient rivets to connect the connection angles to the floorbeams; as it has been found, in practice, that under the actual conditions of loading these rivets become loose before any others in the bridge, showing that they are subjected to secondary stresses whose magnitude cannot be calculated. The number of rivets required is found in the ordinary way, by dividing the load coming from the stringer by the value of one rivet. The critical value of one of these rivets connecting the angles to the floorbeam is usually the value in single shear.

The design of stringers and stringer connections will be treated in subsequent Sections.

FLOORBEAMS

30. Shapes Used for Floorbeams.—I beams are sometimes used for floorbeams of highway bridges, but in railroad bridges, and frequently in highway bridges, plate girders are used.

31. Floorbeam Connections.—Floorbeams are connected to girders and trusses in almost the same way as stringers are connected to floorbeams, but the actual connection depends on the details in each case.

LATERAL SYSTEMS

32. Lateral Trusses.—The lateral trusses of truss bridges were discussed in *Stresses in Bridge Trusses*, Part 5. The lateral trusses of plate-girder bridges are somewhat different from those of truss bridges, but the same principles apply to both. *It is customary to ignore the stresses due to wind pressure and centrifugal force in the girders when the latter are considered as chord members of lateral trusses.*

The diagonals of lateral trusses are usually called **laterals**.

33. In deck girder bridges, lateral trusses are provided in both the top and bottom flanges. Each has the form represented in Fig. 25. The stresses are found in the same

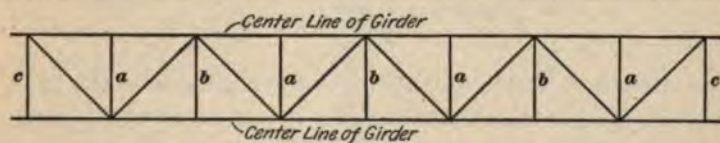


FIG. 25

way as in the Warren truss. As there is only one system of diagonals, the stresses in them will be sometimes tension and sometimes compression, according to the direction in which the wind blows, and they must be designed for this reversal of stress according to the rules given in *Bridge Specifications*. In Fig. 25, the members marked *a* are simply cross-struts to connect the two flanges; those marked *b* are intermediate transverse frames; and those marked *c* are end transverse frames. It is customary to locate the frames so that the diagonals will make an angle of about 45° with the center line of the bridge.

34. In half-through plate-girder bridges, there is but one lateral truss, composed of two systems of diagonals like those described for truss bridges. When the panel length of the bridge is less than one-half the distance center to center of girders, the panels of the lateral trusses are frequently made twice the length of the panel of the bridge, as

represented in Fig. 26. In this figure, a, a are the end floorbeams, and b, c , the intermediate. The diagonals of the two systems intersect each other at the center of the floorbeams marked b , and are attached to each other and to the floorbeams by means of connection plates and angles. The ends of the laterals intersect near the center lines of the girders, at the ends of the floorbeams marked c , and are connected to each other, to the floorbeams, and to the girders by means of connection plates. It is impossible to calculate accurately

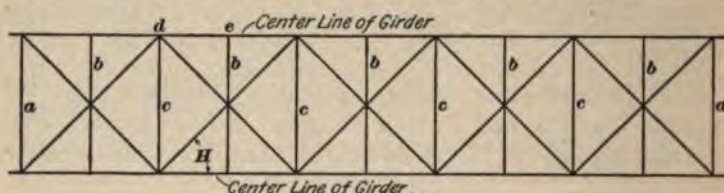


FIG. 26

the stresses in the diagonals of this type of truss: the usual practice is to assume that the shear in any panel of the bridge due to wind pressure or centrifugal force or both is resisted by one diagonal in tension, the other being out of action. For example, if there is no centrifugal force acting on the bridge represented in Fig. 26, and the maximum shear in the panel de due to wind pressure is V_w , then, as the wind may blow in either direction, both diagonals must be designed for a tension equal to $V_w \csc H$.

35. Lateral Connections.—The method of connecting laterals to trusses, girders, and floorbeams depends to a great extent on the connections of stringers to floorbeams and floorbeams to girders and trusses. When the floorbeams of half-through bridges rest on the bottom flanges of the girders, as is frequently the case, the lateral connection is invariably made as represented in Fig. 27. In this figure, aa is a portion of the lower flange of the plate girder; bb , the lateral connection plate resting on top of the flange angles; cc , a portion of the bottom flange of the floorbeam; dd , the bottom flanges of the stringers; and ee , the laterals. When the stringers rest directly on top of the bottom flange

angle of the floorbeam, the laterals are placed below the plates, and fillers *f* of sufficient thickness are placed between the bottom of the stringer and the top of the lateral, as represented at the left of the floorbeam in Fig. 27. When the stringers are riveted to the floorbeams, a short distance, usually not over 7 inches, above the bottom flange angles, the laterals are placed above the connection plates and are connected to the stringers by means of lug or hitch angles *g*,

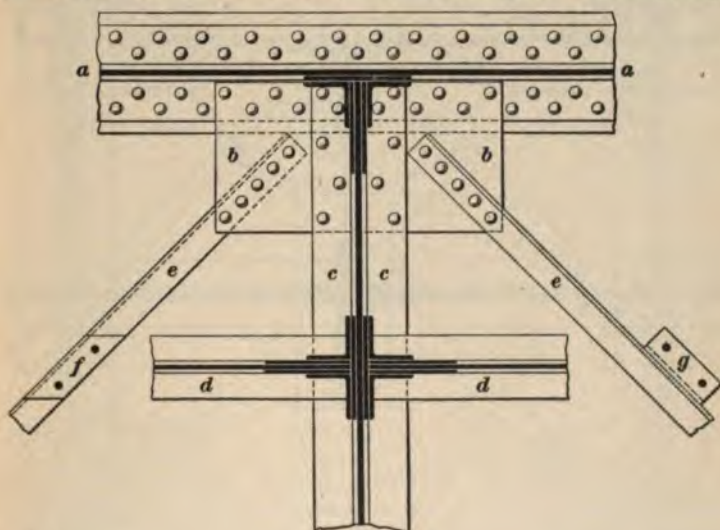


FIG. 27

as represented at the right of the floorbeam in Fig. 27. When the bottoms of stringers are more than 6 or 7 inches above the bottoms of the floorbeams, the connections between the laterals and the stringers are sometimes dispensed with, and the laterals are located in either of the two ways represented in Fig. 27, the fillers *f* and hitch angles *g* being omitted.

36. When the bottoms of the stringers are more than 6 or 7 inches above the bottoms of the floorbeams, and it is desired to maintain the connection between the stringers and the laterals, the arrangement represented in Fig. 28 is

used. The angles a, a are connection or hitch angles riveted to the webs of the girder and floorbeam above the lower flange angles; b, b are the lateral connection plates, which may be either above or below the angles a, a ; and c, c are the laterals, which are invariably placed below the connection plates, as shown, and at such an elevation that they can be riveted directly to the lower flanges of the stringers, as

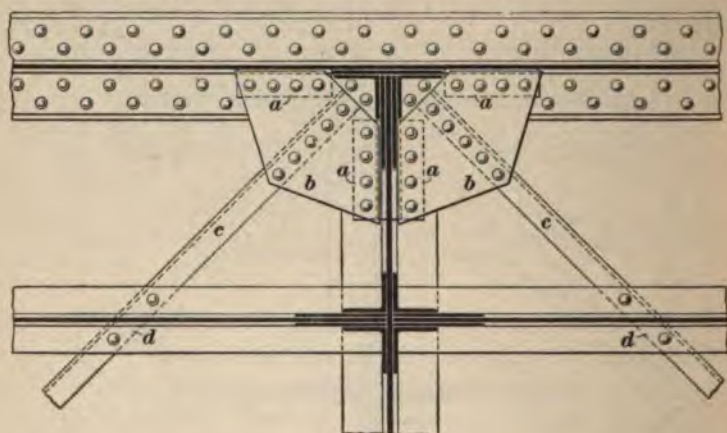


FIG. 28

at d, d , without the necessity of any intermediate filling pieces. There should, as a rule, never be less than four rivets in the connection of the lateral connection plate b to either hitch angle a , nor less than five rivets in the connection of any hitch angle to the web of the girder or floorbeam.

37. The lateral connections of deck bridges are similar to those of half-through bridges. The connection plates are either riveted to the flanges, as represented in Fig. 27, or to hitch angles that are riveted to the webs of the girders below the top flange angles and above the bottom flange angles, as represented in Fig. 28. The latter method is to be preferred in the case of a deck bridge, because it lowers the top lateral system so that it does not interfere with the wooden-floor system, as sometimes happens when the lateral connection

plates are riveted directly to the outstanding legs of the top flange angles.

38. Transverse Frames.—In both of the cases mentioned in the preceding article, a pair of stiffeners is located on the girder at each panel point of the lateral truss, and a transverse strut or frame is riveted to the stiffeners. Fig. 29 represents a cross-section of a deck-girder bridge and an end

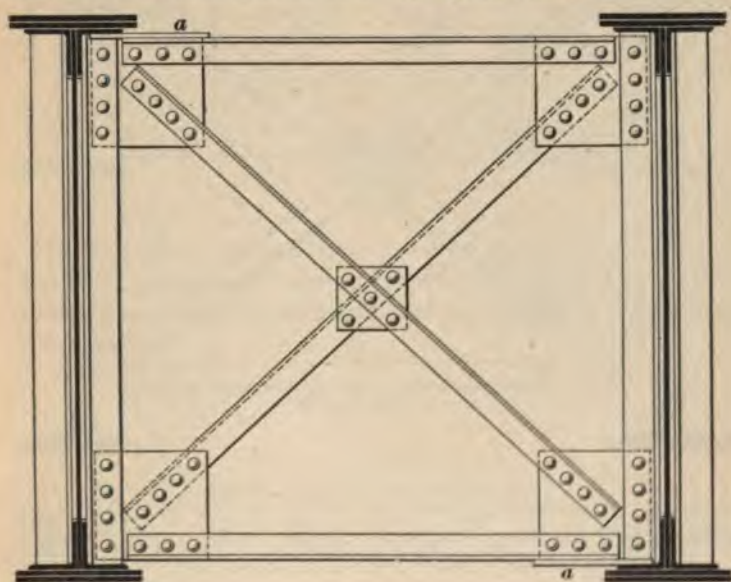


FIG. 29

transverse frame. The lateral connection plates *a, a* are riveted directly to the outstanding legs of the flange angles, and the frame is made deep enough to fit between the connection plates. Fig. 30 also represents a cross-section of a deck plate-girder bridge and an end transverse frame. The lateral connection plates in this case are riveted to the hitch angles, and the frame is made deep enough to fit between the connection plates. In both of these figures, the same detail is used for the intermediate transverse frames as for the end frames, with the exception that but one diagonal is provided. Where only a transverse strut is desired, both diagonals are omitted.

When deck-plate girders are very shallow, say less than 3 feet deep, the transverse frames are sometimes composed of a solid web and two angles, as represented in Fig. 31, or

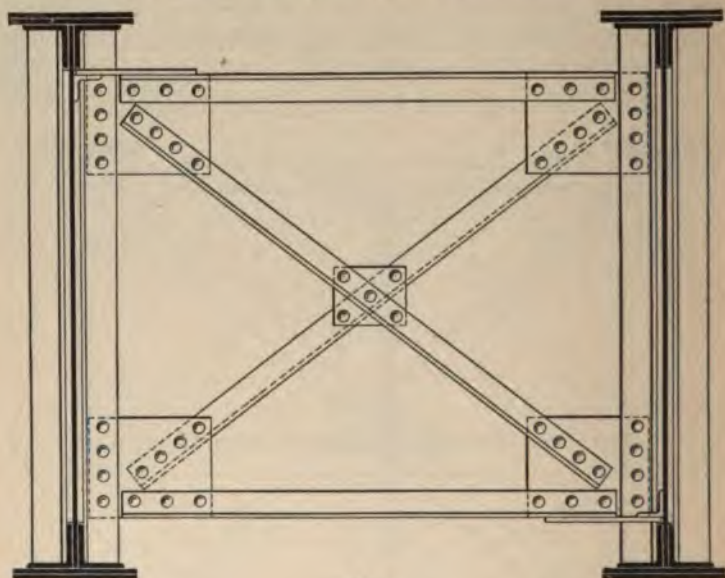


FIG. 30

are latticed, as represented in Fig. 32; in this case the lower lateral truss is usually omitted. When these types of trans-

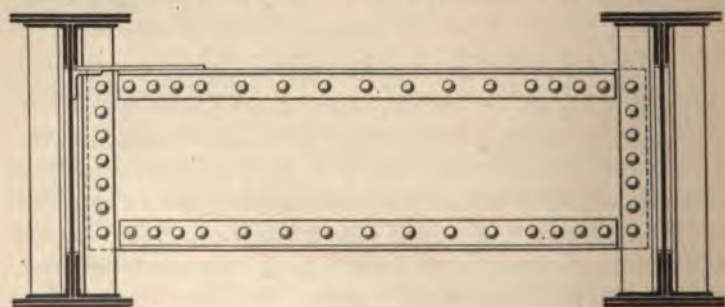


FIG. 31

verse frames are used, the end and intermediate frames are made the same. The frames represented in Figs. 31 and 32

are used also to connect the ends of end stringers to each other and to the girders or trusses when there are no end floorbeams.

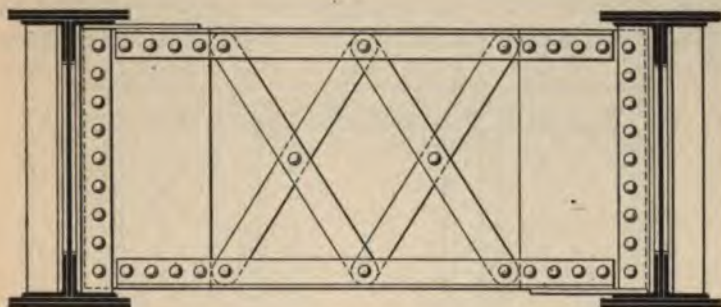


FIG. 32

39. Portals and Transverse Struts.—Portals and transverse struts and frames of through bridges are arranged to conform to other details, and will be treated in connection with design.

BEARINGS

BEDPLATES AND ROCKERS

40. Bedplates.—Where trusses or girders rest on the bridge seat or abutment, large steel plates, called **bedplates**, are placed on the masonry so as to distribute the pressure evenly. The required area of bearing is found by dividing the reaction by the allowable intensity of pressure. In short-span bridges, say less than 75 feet, the reaction is sufficiently distributed by bedplates $\frac{3}{4}$ or 1 inch thick.

41. Rockers.—For spans longer than 75 feet, the method of distribution given in Art. 40 is unsatisfactory, as the greater deflection of the span makes the contact between the bedplate and masonry somewhat imperfect, throwing more load on the front than on the back of the bedplate. To distribute the load more evenly over the masonry, **rocker bearings** are used. A rocker bearing is represented in

outline in Fig. 33. The girder or truss rests on the upper half *a* of a pedestal, in the bottom of which is a semicylindrical hole that just fits a pin placed at right angles to the truss or girder. This pin is supported by the lower half *b* of the pedestal, which rests on the masonry. When this device

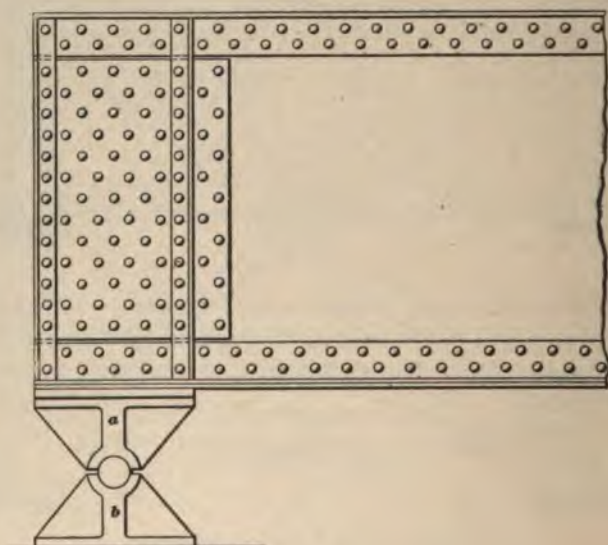


FIG. 33

is used, the load is always concentrated on the pin, and the upper half rocks slightly back and forth as the truss or girder deflects; the load being always concentrated on the pin, which is at the center of the pedestal, the pressure on the masonry is always evenly distributed.

ROLLERS

42. Changes of temperature cause bridges to expand and contract. The difference between the lengths of the bridge at the highest and lowest temperatures may amount to 1 inch in 100 feet. For spans less than about 75 feet, it is assumed that the bridge will adjust itself to these changes, and the top of the bedplate and bottom of the bridge at one end

(commonly called the **expansion end**) are planed smooth so that the end can slide back and forth more easily. For spans of greater length, the load is so great that the ends slide with difficulty, and it is necessary to place rollers, called **expansion rollers**, under one end, and a bedplate between the rollers and the masonry, as shown in Fig. 34. When the

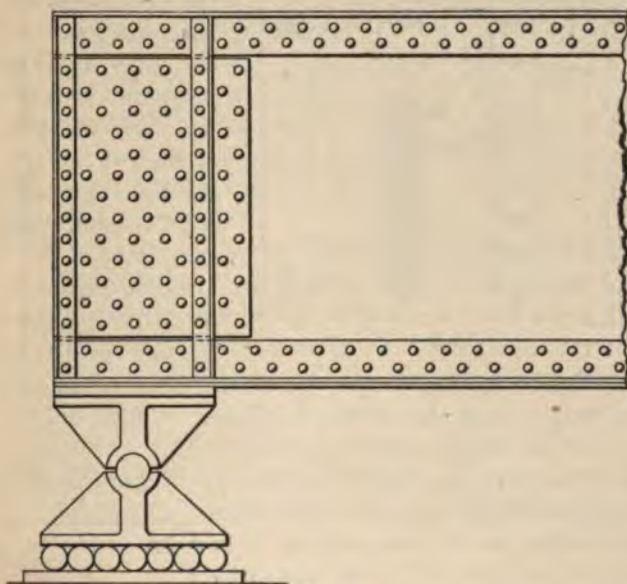


FIG. 34

truss expands or contracts, the end under which the rollers are located moves back and forth on the rollers. These rollers are usually made from 3 to 6 inches in diameter.

The design of pedestals and rollers will be treated elsewhere.

ANCHOR BOLTS

43. Both ends of every truss or girder are fastened to the masonry by means of bolts, called **anchor bolts**. Fig. 35 shows four kinds of anchor bolts that are commonly used. For each of these, a hole about 12 inches deep is drilled in the masonry, the bolt is dropped into this hole,

and the rest of the hole is then filled with sulphur, lead, or cement. The bolts shown at *a* and *b* are sometimes called **rag** or **dog bolts**; that shown at *c* is simply a screw bolt. In the form shown at *d*, a wedge is inserted in the end of

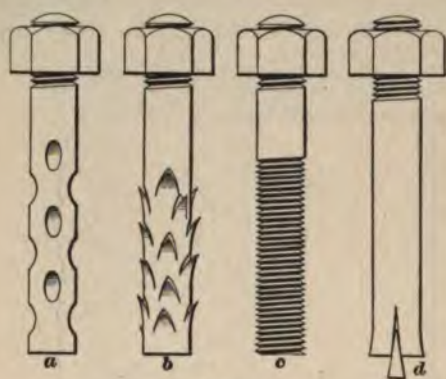


FIG. 35

the bolt, and this end is placed in the hole; the bolt is then struck hard on top to force out the sides at the bottom, so that they will grip the masonry firmly. The rest of the hole is then filled with sulphur, lead, or cement.

At the expansion end of the bridge, the holes in the pedestal or bottom flange through which the anchor bolts must pass are made longer than the width, as shown in Fig. 36, so that that end can move freely backwards and forwards. The bolts at this end simply serve to keep the bridge from moving sidewise; those at the other end, commonly called the **fixed end**, serve to hold the bridge in place longitudinally as well as laterally.



FIG. 36

CAMBER

44. When a bridge is loaded, it deflects and forms a curve, the center of the span being lower than the ends.

To counteract this tendency, and make the floor nearly level when the bridge is loaded, trusses are built so that they curve slightly upwards when there is no load on the bridge, and assume a position almost horizontal when the load is applied. This upward curve is spoken of as the **camber**.

Trusses are cambered by making the panels of the top chord slightly longer than the panel lengths of the lower chord, the usual allowance being $\frac{1}{8}$ inch for every 10 feet. For example, if the panel length of a truss is 20 feet, the pins or intersections of the center lines in the top chord will be 20 feet $\frac{1}{4}$ inch apart. This gives the truss the appearance shown in Fig. 37 to an exaggerated vertical scale. The joints in the chords are approximately on curves, and the verticals may be considered to be on radii to these curves.

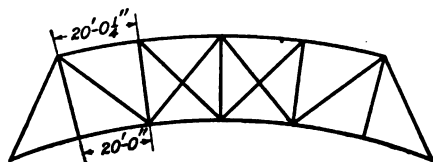


FIG. 37

The actual length of a diagonal of a truss with camber is equal to the length of a diagonal of a truss of the same depth having no camber, and whose panel length is a mean between the actual lengths of the top and bottom chord members. For example, if the height of truss in Fig. 37 is 20 feet, the mean of the lengths of the two chord members in one panel is

$$\frac{1}{2} \times (20 \text{ feet} + 20 \text{ feet } \frac{1}{4} \text{ inch}) = 20 \text{ feet } \frac{1}{8} \text{ inch}$$

and the actual length of a diagonal is

$$\sqrt{(20 \text{ feet})^2 + (20 \text{ feet } \frac{1}{8} \text{ inch})^2} = 28.292 \text{ feet}$$

BRIDGE TABLES

TABLE I
AREAS AND WEIGHTS OF ROUND RODS

Diameter Inches	Area of Section Square Inches	Weight per Foot Pounds	Diameter Inches	Area of Section Square Inches	Weight per Foot Pounds	Diameter Inches	Area of Section Square Inches	Weight per Foot Pounds
$\frac{1}{16}$.003	.010	$1\frac{1}{16}$	1.92	6.52	$3\frac{1}{16}$	7.67	26.08
$\frac{1}{8}$.012	.042	$1\frac{1}{8}$	2.07	7.05	$3\frac{1}{8}$	8.30	28.20
$\frac{3}{16}$.028	.094	$1\frac{1}{4}$	2.24	7.60	$3\frac{1}{4}$	8.95	30.42
$\frac{1}{4}$.049	.167	$1\frac{1}{2}$	2.41	8.18	$3\frac{1}{2}$	9.62	32.71
$\frac{5}{16}$.077	.261	$1\frac{3}{4}$	2.58	8.77	$3\frac{3}{4}$	10.32	35.09
$\frac{3}{8}$.110	.375	$1\frac{7}{8}$	2.76	9.39	$3\frac{7}{8}$	11.04	37.56
$\frac{7}{16}$.150	.511	$1\frac{15}{16}$	2.95	10.02	$3\frac{15}{16}$	11.79	40.10
$\frac{1}{2}$.196	.667	2	3.14	10.68	4	12.57	42.73
$\frac{5}{8}$.248	.845	$2\frac{1}{16}$	3.34	11.36	$4\frac{1}{16}$	13.36	45.44
$\frac{3}{4}$.307	1.04	$2\frac{1}{8}$	3.55	12.06	$4\frac{1}{8}$	14.19	48.24
$\frac{7}{8}$.371	1.26	$2\frac{1}{4}$	3.76	12.78	$4\frac{1}{4}$	15.03	51.11
$\frac{15}{16}$.442	1.50	$2\frac{1}{2}$	3.98	13.52	$4\frac{1}{2}$	15.90	54.07
$1\frac{1}{16}$.518	1.76	$2\frac{3}{16}$	4.20	14.28	$4\frac{3}{16}$	16.80	57.12
$1\frac{1}{8}$.601	2.04	$2\frac{1}{2}$	4.43	15.07	$4\frac{1}{2}$	17.72	60.25
$1\frac{1}{4}$.690	2.35	$2\frac{5}{8}$	4.67	15.80	$4\frac{5}{8}$	18.66	63.46
1	.785	2.67	$2\frac{3}{4}$	4.91	16.69	5	19.63	66.76
$1\frac{1}{16}$.887	3.01	$2\frac{7}{8}$	5.16	17.53	$5\frac{1}{16}$	20.63	70.14
$1\frac{1}{8}$.994	3.38	$2\frac{1}{2}$	5.41	18.40	$5\frac{1}{8}$	21.65	73.60
$1\frac{3}{16}$	1.11	3.77	$2\frac{1}{4}$	5.67	19.29	$5\frac{3}{16}$	22.69	77.15
$1\frac{1}{2}$	1.23	4.17	$2\frac{1}{2}$	5.94	20.20	$5\frac{1}{2}$	23.76	80.77
$1\frac{5}{16}$	1.35	4.60	$2\frac{3}{8}$	6.21	21.12	$5\frac{5}{16}$	24.85	84.49
$1\frac{3}{8}$	1.48	5.05	$2\frac{1}{2}$	6.49	22.07	$5\frac{3}{8}$	25.97	88.29
$1\frac{7}{16}$	1.62	5.52	$2\frac{5}{8}$	6.78	23.04	$5\frac{7}{16}$	27.11	92.17
$1\frac{1}{2}$	1.77	6.01	3	7.07	24.03	6	28.27	96.14

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TABLE II
AREAS AND WEIGHTS OF SQUARE RODS

Side of Square Inches	Area of Section Square Inches	Weight per Foot Pounds	Side of Square Inches	Area of Section Square Inches	Weight per Foot Pounds	Side of Square Inches	Area of Section Square Inches	Weight per Foot Pounds
$\frac{1}{16}$.004	.013	$1\frac{9}{16}$	2.44	8.30	$3\frac{1}{8}$	9.77	33.20
$\frac{1}{8}$.016	.053	$1\frac{1}{2}$	2.64	8.68	$3\frac{1}{4}$	10.56	35.92
$\frac{3}{16}$.035	.120	$1\frac{11}{16}$	2.85	9.68	$3\frac{3}{8}$	11.39	38.73
$\frac{1}{4}$.062	.213	$1\frac{1}{4}$	3.06	10.41	$3\frac{1}{2}$	12.25	41.65
$\frac{5}{16}$.098	.332	$1\frac{3}{8}$	3.29	11.17	$3\frac{5}{8}$	13.14	44.68
$\frac{3}{8}$.141	.478	$1\frac{5}{8}$	3.52	11.95	$3\frac{7}{8}$	14.06	47.82
$\frac{7}{16}$.191	.651	$1\frac{3}{4}$	3.75	12.76	4	15.02	51.05
$\frac{1}{2}$.250	.850	2	4.00	13.60		16.00	54.40
$\frac{9}{16}$.316	1.08	$2\frac{1}{16}$	4.25	14.46	$4\frac{1}{8}$	17.02	57.85
$\frac{5}{8}$.391	1.33	$2\frac{1}{8}$	4.52	15.35	$4\frac{1}{4}$	18.06	61.41
$\frac{11}{16}$.473	1.61	$2\frac{1}{4}$	4.79	16.27	$4\frac{3}{8}$	19.14	65.08
$\frac{3}{4}$.562	1.91	$2\frac{1}{2}$	5.06	17.22	$4\frac{1}{2}$	20.25	68.85
$\frac{13}{16}$.660	2.25	$2\frac{3}{8}$	5.35	18.19	$4\frac{5}{8}$	21.39	72.73
$\frac{7}{8}$.766	2.60	$2\frac{1}{2}$	5.64	19.18	$4\frac{3}{4}$	22.56	76.71
$\frac{15}{16}$.879	2.99	$2\frac{7}{8}$	5.94	20.20	$4\frac{7}{8}$	23.77	80.81
1	1.00	3.40	$2\frac{1}{2}$	6.25	21.25	5	25.00	85.00
$1\frac{1}{16}$	1.13	3.84	$2\frac{9}{16}$	6.57	22.33	$5\frac{1}{8}$	26.27	89.30
$1\frac{1}{8}$	1.27	4.30	$2\frac{5}{8}$	6.89	23.43	$5\frac{1}{4}$	27.56	93.72
$1\frac{3}{16}$	1.41	4.80	$2\frac{11}{16}$	7.22	24.56	$5\frac{3}{8}$	28.89	98.23
$1\frac{1}{4}$	1.56	5.31	$2\frac{1}{2}$	7.56	25.71	$5\frac{1}{2}$	30.25	102.8
$1\frac{5}{16}$	1.72	5.86	$2\frac{3}{4}$	7.91	26.90	$5\frac{5}{8}$	31.64	107.6
$1\frac{3}{8}$	1.89	6.43	$2\frac{7}{8}$	8.27	28.10	$5\frac{3}{4}$	33.06	112.4
$1\frac{7}{16}$	2.07	7.03	$2\frac{15}{16}$	8.63	29.34	$5\frac{7}{8}$	34.52	117.4
$1\frac{1}{2}$	2.25	7.65	3	9.00	30.60	6	36.00	122.4

BRIDGE TABLES

3

TABLE III
UPSET SCREW ENDS FOR ROUND AND SQUARE RODS



Round Rods				Square Rods			
Diameter of Rod Inches d	Diameter of Screw Inches D	Length of Screw Inches L	Additional Length to Form Upset Inches U	Side of Square Inches d	Diameter of Screw Inches D	Length of Screw Inches L	Additional Length to Form Upset Inches U
$\frac{3}{8}$	1	4	$3\frac{7}{8}$	$\frac{3}{8}$	$1\frac{1}{8}$	4	$3\frac{1}{2}$
$\frac{7}{8}$	$1\frac{1}{2}$	4	5	$\frac{7}{8}$	$1\frac{1}{2}$	4	4
1	$1\frac{5}{8}$	4	$4\frac{3}{4}$	1	$1\frac{1}{2}$	4	4
$1\frac{1}{8}$	$1\frac{3}{4}$	4	$3\frac{1}{2}$	$1\frac{1}{8}$	$1\frac{5}{8}$	$4\frac{1}{2}$	$4\frac{1}{2}$
$1\frac{1}{4}$	$1\frac{7}{8}$	$4\frac{1}{2}$	$3\frac{5}{8}$	$1\frac{1}{4}$	$1\frac{7}{8}$	$4\frac{1}{2}$	$4\frac{1}{2}$
$1\frac{3}{8}$	$1\frac{1}{2}$	$4\frac{1}{2}$	$3\frac{1}{2}$	$1\frac{3}{8}$	2	5	$4\frac{1}{8}$
$1\frac{1}{2}$	2	5	$4\frac{5}{8}$	$1\frac{1}{2}$	$2\frac{1}{2}$	5	$4\frac{3}{4}$
$1\frac{5}{8}$	$2\frac{1}{8}$	5	$4\frac{1}{4}$	$1\frac{5}{8}$	$2\frac{3}{8}$	$5\frac{1}{2}$	$4\frac{5}{8}$
$1\frac{3}{4}$	$2\frac{1}{4}$	5	4	$1\frac{3}{4}$	$2\frac{1}{2}$	$5\frac{1}{2}$	$4\frac{1}{2}$
$1\frac{7}{8}$	$2\frac{3}{8}$	$5\frac{1}{2}$	$4\frac{1}{8}$	$1\frac{7}{8}$	$2\frac{3}{4}$	6	$5\frac{1}{8}$
2	$2\frac{1}{2}$	$5\frac{1}{2}$	$3\frac{7}{8}$	2	$2\frac{7}{8}$	6	$4\frac{3}{4}$
$2\frac{1}{8}$	$2\frac{5}{8}$	$5\frac{1}{2}$	$3\frac{5}{8}$	$2\frac{1}{8}$	3	6	$4\frac{3}{8}$
$2\frac{1}{4}$	$2\frac{7}{8}$	6	$4\frac{5}{8}$	$2\frac{1}{4}$	$3\frac{1}{4}$	$6\frac{1}{2}$	$5\frac{1}{8}$
$2\frac{3}{8}$	3	6	$4\frac{3}{4}$	$2\frac{3}{8}$	$3\frac{1}{2}$	7	$6\frac{1}{8}$
$2\frac{1}{2}$	$3\frac{1}{8}$	$6\frac{1}{2}$	$4\frac{3}{8}$	$2\frac{1}{2}$	$3\frac{5}{8}$	8	$6\frac{1}{2}$
$2\frac{5}{8}$	$3\frac{1}{4}$	$6\frac{1}{2}$	$4\frac{1}{4}$	$2\frac{5}{8}$	$3\frac{7}{8}$	8	$6\frac{3}{4}$
$2\frac{3}{4}$	$3\frac{3}{8}$	7	$4\frac{1}{2}$	$2\frac{3}{4}$	4	8	6
$2\frac{7}{8}$	$3\frac{5}{8}$	8	$5\frac{1}{2}$	$2\frac{7}{8}$	$4\frac{1}{4}$	9	8
3	$3\frac{3}{4}$	8	$5\frac{1}{4}$	3	$4\frac{1}{2}$	9	$7\frac{1}{2}$

TABLE IV
SIZES AND WEIGHTS OF HEXAGON NUTS



Diameter of Screw Inches	Thickness of Nut <i>T</i> Inches	Short Diameter <i>d</i> Inches	Long Diameter <i>D</i> Inches	Weight of 100 Nuts Pounds
$\frac{1}{2}$	$\frac{1}{2}$	1	1.15	9.8
$\frac{5}{8}$	$\frac{3}{4}$	$1\frac{1}{4}$	1.44	22.9
$\frac{3}{4}$	$\frac{7}{8}$	$1\frac{1}{2}$	1.73	39
$\frac{7}{8}$	1	$1\frac{3}{8}$	1.88	50
1	$1\frac{1}{8}$	$1\frac{3}{4}$	2.02	64
$1\frac{1}{8}$	$1\frac{1}{4}$	2	2.31	96
$1\frac{1}{4}$	$1\frac{3}{8}$	$2\frac{1}{4}$	2.60	134
$1\frac{3}{8}$	$1\frac{1}{2}$	$2\frac{1}{2}$	2.89	180
$1\frac{1}{2}$	$1\frac{5}{8}$	$2\frac{3}{4}$	3.18	235
$1\frac{5}{8}$	$1\frac{3}{4}$	3	3.46	300
$1\frac{3}{4}$	$1\frac{7}{8}$	$3\frac{1}{4}$	3.75	370
$1\frac{7}{8}$	2	$3\frac{1}{2}$	4.04	460
2	2	$3\frac{1}{2}$	4.04	450
$2\frac{1}{4}$	$2\frac{1}{4}$	$3\frac{3}{4}$	4.33	560
$2\frac{1}{2}$	$2\frac{1}{2}$	$4\frac{1}{4}$	4.91	810
$2\frac{3}{4}$	$2\frac{3}{4}$	$4\frac{1}{2}$	5.20	980
3	3	$4\frac{3}{4}$	5.48	1,150
$3\frac{1}{4}$	$3\frac{1}{4}$	5	5.77	1,340
$3\frac{1}{2}$	$3\frac{1}{2}$	$5\frac{1}{4}$	6.06	1,580

BRIDGE TABLES

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TABLE V
APPROXIMATE MAXIMUM LENGTHS OF STEEL PLATES, IN FEET

Width Inches	Thickness, in Inches									
	$\frac{1}{2}$	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2	$2\frac{1}{2}$	3	$3\frac{1}{2}$
10	45	70	70	70	70	70	70	70	70	70
20	45	90	90	80	80	75	75	65	60	50
30	*	90	90	80	60	50	50	40	40	30
40	*	80	90	80	65	50	40	35	30	25
50	28	40	40	40	34	30	28	25	22	22
60	25	38	40	33	30	26	24	22	18	18
70	22	35	34	29	26	22	20	19	17	16
80	20	30	30	25	23	19	19	17	15	15
90	13	27	27	21	18	17	16	14	13	13
100	8	16	20	18	16	15	14	12	11	11
110	*	*	14	13	13	11	10	10	9	*
120	*	*	*	10	10	10	10	10	*	*

*Plates having these dimensions are not ordinarily handled by the rolling mills.

TABLE VI
AREAS OF SECTIONS OF STEEL PLATES,
IN SQUARE INCHES

Width Inches	Thickness, in Inches							
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$
$\frac{1}{4}$.0156	.0313	.0469	.0625	.0781	.0938	.1094	.1250
$\frac{3}{8}$.0313	.0625	.0938	.1250	.1563	.1875	.2188	.2500
$\frac{1}{2}$.0469	.0938	.1406	.1875	.2344	.2813	.3281	.3750
1	.0625	.1250	.1875	.2500	.3125	.3750	.4375	.5000
2	.1250	.2500	.3750	.5000	.6250	.7500	.8750	1.000
3	.1875	.3750	.5625	.750	.9375	1.125	1.313	1.500
4	.2500	.5000	.7500	1.000	1.250	1.500	1.750	2.000
5	.3125	.6250	.9375	1.250	1.563	1.875	2.188	2.500
6	.3750	.7500	1.125	1.500	1.875	2.250	2.625	3.000
7	.4375	.8750	1.313	1.750	2.188	2.625	3.063	3.500
8	.5000	1.000	1.500	2.000	2.500	3.000	3.500	4.000
9	.5625	1.125	1.688	2.250	2.813	3.375	3.938	4.500
10	.6250	1.250	1.875	2.500	3.125	3.750	4.375	5.000
11	.6875	1.375	2.063	2.750	3.438	4.125	4.813	5.500
12	.7500	1.500	2.250	3.000	3.750	4.500	5.250	6.000
13	.8125	1.625	2.438	3.250	4.063	4.875	5.688	6.500
14	.8750	1.750	2.625	3.500	4.375	5.250	6.125	7.000
15	.9375	1.875	2.813	3.750	4.688	5.625	6.563	7.500
16	1.000	2.000	3.000	4.000	5.000	6.000	7.000	8.000
18	1.125	2.250	3.375	4.500	5.625	6.750	7.875	9.000
20	1.250	2.500	3.750	5.000	6.250	7.500	8.750	10.000
30	1.875	3.750	5.625	7.500	9.375	11.250	13.125	15.000
40	2.500	5.000	7.500	10.000	12.500	15.000	17.500	20.000
50	3.125	6.250	9.375	12.500	15.625	18.750	21.875	25.000
60	3.750	7.500	11.250	15.000	18.750	22.500	26.250	30.000
70	4.375	8.750	13.125	17.500	21.875	26.250	30.625	35.000
80	5.000	10.000	15.000	20.000	25.000	30.000	35.000	40.000
90	5.625	11.250	16.875	22.500	28.125	33.750	39.375	45.000
100	6.250	12.500	18.750	25.000	31.250	37.500	43.750	50.000

BRIDGE TABLES

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TABLE VI—(Continued)
AREAS OF SECTIONS OF STEEL PLATES,
IN SQUARE INCHES

Width Inches	Thickness, in Inches							
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
$\frac{1}{4}$.1406	.1563	.1719	.1875	.2031	.2188	.2344	.2500
$\frac{1}{2}$.2813	.3125	.3438	.3750	.4063	.4375	.4688	.5000
$\frac{3}{4}$.4219	.4688	.5156	.5625	.6094	.6563	.7031	.7500
1	.5625	.6250	.6875	.7500	.8125	.8750	.9375	1.0000
2	1.125	1.250	1.375	1.500	1.625	1.750	1.875	2.000
3	1.688	1.875	2.063	2.250	2.438	2.625	2.813	3.000
4	2.250	2.500	2.750	3.000	3.250	3.500	3.750	4.000
5	2.813	3.125	3.438	3.750	4.063	4.375	4.688	5.000
6	3.375	3.750	4.125	4.500	4.875	5.250	5.625	6.000
7	3.938	4.375	4.813	5.250	5.688	6.125	6.563	7.000
8	4.500	5.000	5.500	6.000	6.500	7.000	7.500	8.000
9	5.063	5.625	6.188	6.750	7.313	7.875	8.438	9.000
10	5.625	6.250	6.875	7.500	8.125	8.750	9.375	10.000
11	6.188	6.875	7.563	8.250	8.938	9.625	10.313	11.000
12	6.750	7.500	8.250	9.000	9.750	10.500	11.250	12.000
13	7.313	8.125	8.938	9.750	10.563	11.375	12.188	13.000
14	7.875	8.750	9.625	10.500	11.375	12.250	13.125	14.000
15	8.438	9.375	10.313	11.250	12.188	13.125	14.063	15.000
16	9.000	10.000	11.000	12.000	13.000	14.000	15.000	16.000
18	10.125	11.250	12.375	13.500	14.625	15.750	16.880	18.000
20	11.250	12.500	13.750	15.000	16.250	17.500	18.750	20.000
30	16.875	18.750	20.625	22.500	24.375	26.250	28.125	30.000
40	22.500	25.000	27.500	30.000	32.500	35.000	37.500	40.000
50	28.125	31.250	34.375	37.500	40.625	43.750	46.875	50.000
60	33.750	37.500	41.250	45.000	48.750	52.500	56.250	60.000
70	39.375	43.750	48.125	52.500	56.875	61.250	65.625	70.000
80	45.000	50.000	55.000	60.000	65.000	70.000	75.000	80.000
90	50.625	56.250	61.875	67.500	73.125	78.750	84.375	90.000
100	56.250	62.500	68.750	75.000	81.250	87.500	93.750	100.000

TABLE VII
WEIGHT OF STEEL PLATES, IN POUNDS PER LINEAR FOOT

Width Inches	Thickness, in Inches							
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{2}$	$\frac{5}{16}$	$\frac{3}{4}$	$\frac{7}{8}$	1
$\frac{1}{4}$.0531	.1063	.1594	.2125	.2656	.3188	.3719	.4250
$\frac{1}{2}$.1063	.2125	.3188	.4250	.5313	.6375	.7438	.8500
$\frac{3}{4}$.1594	.3188	.4781	.6375	.7969	.9563	1.116	1.275
1	.2125	.4250	.6375	.8500	1.063	1.275	1.488	1.700
2	.4250	.850	1.275	1.700	2.125	2.550	2.975	3.400
3	.6375	1.275	1.913	2.550	3.188	3.825	4.463	5.100
4	.8500	1.700	2.550	3.400	4.250	5.100	5.950	6.800
5	1.063	2.125	3.188	4.250	5.313	6.375	7.438	8.500
6	1.275	2.550	3.825	5.100	6.375	7.650	8.925	10.200
7	1.488	2.975	4.463	5.950	7.438	8.925	10.413	11.900
8	1.700	3.400	5.100	6.800	8.500	10.200	11.900	13.600
9	1.913	3.825	5.738	7.650	9.563	11.480	13.390	15.300
10	2.125	4.250	6.375	8.500	10.630	12.750	14.880	17.000
11	2.338	4.675	7.013	9.350	11.690	14.030	16.360	18.700
12	2.550	5.100	7.650	10.200	12.750	15.300	17.850	20.400
13	2.763	5.525	8.288	11.050	13.810	16.580	19.340	22.100
14	2.975	5.950	8.925	11.900	14.880	17.850	20.830	23.800
15	3.188	6.375	9.563	12.750	15.940	19.130	22.310	25.500
16	3.400	6.800	10.200	13.600	17.000	20.400	23.800	27.200
18	3.825	7.650	11.480	15.300	19.130	22.950	26.780	30.600
20	4.250	8.500	12.750	17.000	21.250	25.500	29.750	34.000
30	6.375	12.750	19.130	25.500	31.880	38.250	44.630	51.000
40	8.500	17.000	25.500	34.000	42.500	51.000	59.500	68.000
50	10.630	21.250	31.880	42.500	53.130	63.750	74.380	85.000
60	12.750	25.500	38.250	51.000	63.750	76.500	89.250	102.000
70	14.880	29.750	44.630	59.500	74.380	89.250	104.100	119.000
80	17.000	34.000	51.000	68.000	85.000	102.000	119.000	136.000
90	19.130	38.250	57.380	76.500	95.630	114.800	133.900	153.000
100	21.250	42.500	63.750	85.000	106.300	127.500	148.800	170.000

BRIDGE TABLES

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TABLE VII—(Continued)

WEIGHT OF STEEL PLATES, IN POUNDS PER LINEAR FOOT

Width Inches	Thickness, in Inches							
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	1
$\frac{1}{8}$.4781	.5313	.5844	.6375	.6906	.7438	.7969	.85
$\frac{1}{4}$.9563	1.063	1.169	1.275	1.381	1.488	1.594	1.70
$\frac{3}{8}$	1.434	1.594	1.753	1.913	2.072	2.231	2.391	2.55
1	1.913	2.125	2.338	2.550	2.763	2.975	3.188	3.40
2	3.825	4.250	4.675	5.100	5.525	5.950	6.375	6.80
3	5.738	6.375	7.013	7.650	8.288	8.925	9.563	10.20
4	7.650	8.500	9.350	10.20	11.05	11.90	12.75	13.60
5	9.563	10.63	11.69	12.75	13.81	14.88	15.94	17.00
6	11.48	12.75	14.03	15.30	16.58	17.85	19.13	20.40
7	13.39	14.88	16.36	17.85	19.34	20.83	22.31	23.80
8	15.30	17.00	18.70	20.40	22.10	23.80	25.50	27.20
9	17.21	19.13	21.04	22.95	24.86	26.78	28.69	30.60
10	19.13	21.25	23.38	25.50	27.63	29.75	31.88	34.00
11	21.04	23.38	25.71	28.05	30.39	32.73	35.06	37.40
12	22.95	25.50	28.05	30.60	33.15	35.70	38.25	40.80
13	24.86	27.63	30.39	33.15	35.91	38.68	41.44	44.20
14	26.78	29.75	32.73	35.70	38.68	41.65	44.63	47.60
15	28.69	31.88	35.06	38.25	41.44	44.63	47.81	51.00
16	30.60	34.00	37.40	40.80	44.20	47.60	51.00	54.40
18	34.43	38.25	42.08	45.90	49.73	53.55	57.38	61.20
20	38.25	42.50	46.75	51.00	55.25	59.50	63.75	68.00
30	57.38	63.75	70.13	76.50	82.88	89.25	95.63	102.0
40	76.50	85.00	93.50	102.0	110.5	119.0	127.5	136.0
50	95.63	106.3	116.9	127.5	138.1	148.8	159.4	170.0
60	114.8	127.5	140.3	153.0	165.8	178.5	191.3	204.0
70	133.9	148.8	163.6	178.5	193.4	208.3	223.1	238.0
80	153.0	170.0	187.0	204.0	221.0	238.0	255.0	272.0
90	172.1	191.3	210.4	229.5	248.6	267.8	286.9	306.0
100	191.3	212.5	233.8	255.0	276.3	297.5	318.8	340.0

TABLE VIII
MOMENTS OF INERTIA OF RECTANGULAR SECTIONS



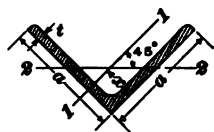
Width Inches	Thickness of Rectangle, in Inches						
	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$
2	.17	.21	.25	.29	.33	.38	.42
3	.56	.70	.84	.98	1.13	1.27	1.41
4	1.33	1.67	2.00	2.33	2.67	3.00	3.33
5	2.60	3.26	3.91	4.56	5.21	5.86	6.51
6	4.50	5.63	6.75	7.88	9.00	10.13	11.25
7	7.15	8.93	10.72	12.51	14.29	16.08	17.86
8	10.67	13.33	16.00	18.67	21.33	24.00	26.67
9	15.19	18.98	22.78	26.58	30.38	34.17	37.97
10	20.83	26.04	31.25	36.46	41.67	46.87	52.08
11	27.73	34.66	41.59	48.53	55.46	62.39	69.32
12	36.00	45.00	54.00	63.00	72.00	81.00	90.00
13	45.77	57.21	68.66	80.10	92.54	104.98	117.43
14	57.17	71.46	85.75	100.04	114.33	128.63	142.92
15	70.31	87.89	105.47	123.05	140.63	158.20	175.78
16	85.33	106.67	128.00	149.33	170.67	192.00	213.33
17	102.35	127.94	153.53	179.12	204.71	230.30	255.89
18	121.50	151.88	182.25	212.63	243.00	273.38	303.75
19	142.90	178.62	214.34	250.07	285.79	321.52	357.24
20	166.67	208.33	250.00	291.67	333.33	375.00	416.67
22	221.83	277.29	332.75	388.21	443.67	499.13	554.58
24	288.00	360.00	432.00	504.00	576.00	648.00	720.00
26	366.17	457.71	549.25	640.79	732.33	823.88	915.42
28	457.33	571.67	686.00	800.33	914.67	1,029.00	1,143.33
30	562.50	703.13	843.75	984.38	1,125.00	1,265.63	1,406.25
32	682.67	853.33	1,024.00	1,194.67	1,365.33	1,536.00	1,706.67
34	818.83	1,023.54	1,228.25	1,432.96	1,637.67	1,842.38	2,047.08
36	972.00	1,215.00	1,458.00	1,701.00	1,944.00	2,187.00	2,430.00
38	1,143.17	1,428.96	1,714.75	2,000.54	2,286.33	2,572.13	2,857.92
40	1,333.33	1,666.67	2,000.00	2,333.33	2,666.67	3,000.00	3,333.33
42	1,543.50	1,929.38	2,315.25	2,701.13	3,087.00	3,472.88	3,858.75
44	1,774.67	2,218.33	2,662.00	3,105.67	3,549.33	3,993.00	4,436.67
46	2,027.83	2,534.79	3,041.75	3,548.71	4,055.67	4,562.63	5,069.58
48	2,304.00	2,880.00	3,450.00	4,032.00	4,608.00	5,184.00	5,760.00
52	2,929.33	3,661.67	4,394.00	5,126.33	5,858.67	6,591.00	7,323.33
56	3,658.67	4,573.33	5,488.00	6,402.67	7,317.33	8,232.00	9,146.67
60	4,500.00	5,625.00	6,750.00	7,875.00	9,000.00	10,125.00	11,250.00

TABLE VIII—(Continued)
MOMENTS OF INERTIA OF RECTANGULAR SECTIONS



Width Inches	Thickness of Rectangle, in Inches					
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{2}$	$\frac{5}{8}$	1
2	.46	.50	.54	.58	.63	.67
3	1.55	1.69	1.83	1.97	2.11	2.25
4	3.67	4.00	4.33	4.67	5.00	5.33
5	7.16	7.81	8.46	9.11	9.77	10.42
6	12.38	13.50	14.63	15.75	16.88	18.00
7	19.65	21.44	23.22	25.01	26.80	28.58
8	29.33	32.00	34.67	37.33	40.00	42.67
9	41.77	45.56	49.36	53.16	56.95	60.75
10	57.29	62.50	67.71	72.92	78.13	83.33
11	76.26	83.19	90.12	97.05	103.98	110.92
12	99.00	108.00	117.00	126.00	135.00	144.00
13	125.87	137.31	148.75	160.20	171.64	183.08
14	157.21	171.50	185.79	200.08	214.38	228.67
15	193.36	210.94	228.52	246.09	263.67	281.25
16	234.67	256.00	277.33	298.67	320.00	341.33
17	281.47	307.06	332.65	358.24	383.83	409.42
18	334.13	364.50	394.88	425.25	455.63	486.00
19	392.96	428.69	464.41	500.14	535.86	571.58
20	458.33	500.00	541.67	583.33	625.00	666.67
22	610.04	665.50	720.96	776.42	831.87	887.33
24	792.00	864.00	936.00	1,008.00	1,080.00	1,152.00
26	1,006.96	1,098.50	1,190.04	1,281.58	1,373.13	1,464.67
28	1,257.67	1,372.00	1,486.33	1,600.67	1,715.00	1,829.33
30	1,546.88	1,687.50	1,828.13	1,968.75	2,109.38	2,250.00
32	1,877.33	2,048.00	2,218.67	2,389.33	2,560.00	2,730.67
34	2,251.79	2,456.50	2,661.21	2,865.92	3,070.63	3,275.33
36	2,673.00	2,916.00	3,159.00	3,402.00	3,645.00	3,888.00
38	3,143.71	3,429.50	3,715.29	4,001.08	4,286.88	4,572.67
40	3,666.67	4,000.00	4,333.33	4,666.67	5,000.00	5,333.33
42	4,244.63	4,630.50	5,016.38	5,402.25	5,788.13	6,174.00
44	4,880.33	5,324.00	5,767.67	6,211.33	6,655.00	7,098.67
46	5,576.54	6,083.50	6,590.46	7,097.42	7,604.38	8,111.33
48	6,336.00	6,912.00	7,488.00	8,064.00	8,640.00	9,216.00
52	8,055.67	8,788.00	9,520.33	10,252.67	10,985.00	11,717.33
56	10,061.33	10,976.00	11,890.67	12,805.33	13,720.00	14,634.67
60	12,375.00	13,500.00	14,625.00	15,750.00	16,875.00	18,000.00

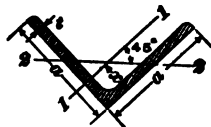
TABLE IX
PROPERTIES OF STANDARD ANGLES HAVING EQUAL LEGS



1	2	3	4	5	6	7	8	9
Widths Inches	Thickness Inches	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity From Back of Flange, Inches	Moment of Inertia, Referred to the Inch Axis 1-1	Section Modulus, Referred to the Inch Axis 1-1	Radius of Gyration, Axis 1-1 Inches	Least Radius of Gyration, Axis 2-2 Inches
$a \times a$	t		A	x	I	Q	r	r
1 × 1	$\frac{1}{8}$.80	.23	.30	.022	.031	.30	.19
1 × 1	$\frac{3}{16}$	1.16	.34	.32	.030	.044	.30	.19
1 × 1	$\frac{1}{2}$	1.49	.44	.34	.037	.056	.29	.19
1½ × 1½	$\frac{1}{8}$	1.02	.30	.36	.044	.049	.38	.24
1½ × 1½	$\frac{3}{16}$	1.47	.43	.38	.061	.071	.38	.24
1½ × 1½	$\frac{1}{2}$	1.91	.56	.40	.077	.091	.37	.24
1½ × 1½	$\frac{5}{16}$	2.32	.68	.42	0.090	.11	.36	.24
1½ × 1½	$\frac{3}{16}$	1.70	.53	.44	.11	.10	.46	.29
1½ × 1½	$\frac{1}{2}$	2.34	.69	.47	.14	.13	.45	.29
1½ × 1½	$\frac{5}{16}$	2.86	.84	.49	.16	.16	.44	.29
1½ × 1½	$\frac{3}{8}$	3.35	.98	.51	.19	.19	.44	.29
1½ × 1½	$\frac{3}{16}$	2.11	.62	.51	.18	.14	.54	.34
1½ × 1½	$\frac{1}{2}$	2.76	.81	.53	.23	.19	.53	.34
1½ × 1½	$\frac{5}{16}$	3.30	1.00	.55	.27	.23	.52	.34
1½ × 1½	$\frac{3}{8}$	3.98	1.17	.57	.31	.26	.51	.34
1½ × 1½	$\frac{7}{16}$	4.56	1.34	.59	.35	.30	.51	.34
2 × 2	$\frac{3}{16}$	2.43	.71	.57	.27	.19	.62	.39
2 × 2	$\frac{1}{2}$	3.10	.94	.59	.35	.25	.61	.39
2 × 2	$\frac{5}{16}$	3.92	1.15	.61	.42	.30	.60	.39
2 × 2	$\frac{3}{8}$	4.62	1.36	.64	.48	.35	.59	.39
2 × 2	$\frac{7}{16}$	5.30	1.56	.66	.54	.40	.59	.38
2½ × 2½	$\frac{3}{16}$	3.1	.90	.69	.55	.30	.78	.49
2½ × 2½	$\frac{1}{2}$	4.0	1.19	.72	.70	.39	.77	.49
2½ × 2½	$\frac{5}{16}$	5.0	1.46	.74	.85	.48	.76	.49
2½ × 2½	$\frac{3}{8}$	5.9	1.73	.76	.98	.57	.75	.48
2½ × 2½	$\frac{7}{16}$	6.8	2.00	.78	1.11	.65	.75	.48
2½ × 2½	$\frac{1}{2}$	7.7	2.25	.81	1.23	.72	.74	.48

TABLE IX—(Continued)

PROPERTIES OF STANDARD ANGLES HAVING EQUAL LEGS



1	2	3	4	5	6	7	8	9
Widths Inches	Thickness Inches	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity From Back of Flange, Inches	Moment of Inertia, Referred to the Inch Axis $I-I$	Section Modulus, Referred to the Inch Axis $I-I$	Radius of Gyration, Axis $I-I$ Inches	Least Radius of Gyration, Axis $Y-Y$ Inches
$a \times a$	t		A	x	I	Q	r	r_y
3×3	$\frac{1}{16}$	4.9	1.44	.84	1.24	.58	.93	.59
3×3	$\frac{1}{8}$	6.0	1.78	.87	1.51	.71	.92	.59
3×3	$\frac{3}{16}$	7.2	2.11	.89	1.76	.83	.91	.58
3×3	$\frac{1}{4}$	8.3	2.43	.91	1.99	.95	.91	.58
3×3	$\frac{5}{16}$	9.4	2.75	.93	2.22	1.07	.90	.58
3×3	$\frac{3}{8}$	10.4	3.06	.95	2.43	1.19	.89	.58
3×3	$\frac{7}{16}$	11.4	3.36	.98	2.62	1.30	.88	.58
3½×3½	$\frac{5}{16}$	7.2	2.09	.99	2.45	.98	1.08	.60
3½×3½	$\frac{3}{8}$	8.4	2.48	1.01	2.87	1.15	1.07	.68
3½×3½	$\frac{7}{16}$	9.8	2.87	1.04	3.26	1.32	1.07	.68
3½×3½	$\frac{1}{2}$	11.1	3.25	1.06	3.64	1.49	1.06	.68
3½×3½	$\frac{9}{16}$	12.3	3.62	1.08	3.99	1.65	1.05	.68
3½×3½	$\frac{5}{8}$	13.5	3.98	1.10	4.33	1.81	1.04	.68
3½×3½	$\frac{11}{16}$	14.8	4.34	1.12	4.65	1.96	1.04	.67
3½×3½	$\frac{3}{4}$	15.9	4.69	1.15	4.96	2.11	1.03	.67
3½×3½	$\frac{13}{16}$	17.1	5.03	1.17	5.25	2.25	1.02	.67
4×4	$\frac{1}{8}$	8.2	2.40	1.12	3.71	1.29	1.24	.79
4×4	$\frac{3}{16}$	9.7	2.86	1.14	4.36	1.52	1.23	.79
4×4	$\frac{1}{4}$	11.2	3.31	1.16	4.97	1.75	1.23	.78
4×4	$\frac{5}{16}$	12.8	3.75	1.18	5.56	1.97	1.22	.78
4×4	$\frac{3}{8}$	14.2	4.18	1.21	6.12	2.19	1.21	.78
4×4	$\frac{7}{16}$	15.7	4.61	1.23	6.66	2.40	1.20	.77
4×4	$\frac{1}{2}$	17.1	5.03	1.25	7.17	2.61	1.19	.77
4×4	$\frac{9}{16}$	18.5	5.44	1.27	7.66	2.81	1.19	.77
4×4	$\frac{5}{8}$	19.9	5.84	1.29	8.14	3.01	1.18	.77
5×5	$\frac{3}{8}$	12.3	3.61	1.39	8.74	2.42	1.56	.90
5×5	$\frac{1}{2}$	14.3	4.18	1.41	10.02	2.79	1.55	.98
5×5	$\frac{5}{8}$	16.2	4.75	1.43	11.25	3.15	1.54	.98

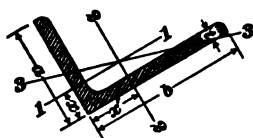
TABLE IX—(Continued)

PROPERTIES OF STANDARD ANGLES HAVING EQUAL LEGS



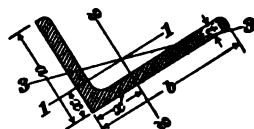
1	2	3	4	5	6	7	8	9
Width Inches	Thickness Inches	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity From Back of Flange, Inches	Moment of Inertia, Referred to the Inch Axis 1-1	Section Modulus, Referred to the Inch Axis 1-1	Radius of Gyration, Axis 1-1 Inches	Least Radius of Gyration, Axis 2-2 Inches
$a \times a$	t		A	x	I	Q	r	r'
5×5	$\frac{3}{16}$	18.1	5.31	1.46	12.44	3.51	1.53	.98
5×5	$\frac{1}{8}$	20.0	5.86	1.48	13.58	3.86	1.52	.97
5×5	$\frac{1}{4}$	21.8	6.42	1.50	14.68	4.20	1.51	.97
5×5	$\frac{1}{2}$	23.6	6.94	1.52	15.74	4.53	1.51	.97
5×5	$\frac{3}{4}$	25.4	7.46	1.55	16.77	4.85	1.50	.97
5×5	$\frac{7}{8}$	27.2	7.99	1.57	17.75	5.17	1.49	.96
5×5	1	28.9	8.50	1.59	18.71	5.49	1.48	.96
5×5	1	30.6	9.00	1.61	19.64	5.80	1.48	.96
6×6	$\frac{3}{16}$	14.9	4.36	1.64	15.39	3.53	1.88	1.19
6×6	$\frac{1}{8}$	17.2	5.06	1.66	17.68	4.07	1.87	1.19
6×6	$\frac{1}{4}$	19.6	5.75	1.68	19.91	4.61	1.86	1.18
6×6	$\frac{3}{8}$	21.9	6.43	1.71	22.07	5.14	1.85	1.18
6×6	$\frac{1}{2}$	24.2	7.11	1.73	24.16	5.66	1.84	1.17
6×6	$\frac{3}{4}$	26.4	7.78	1.75	26.19	6.17	1.83	1.17
6×6	1	28.7	8.44	1.78	28.15	6.66	1.83	1.17
6×6	$\frac{1}{8}$	30.9	9.09	1.80	30.06	7.15	1.82	1.17
6×6	$\frac{1}{4}$	33.1	9.73	1.82	31.92	7.63	1.81	1.16
6×6	$\frac{3}{8}$	35.3	10.37	1.84	33.72	8.11	1.80	1.16
6×6	1	37.4	11.00	1.86	35.46	8.57	1.80	1.16
8×8	$\frac{1}{8}$	26.4	7.75	2.19	48.63	8.37	2.50	1.58
8×8	$\frac{1}{4}$	29.6	8.68	2.21	54.09	9.34	2.50	1.58
8×8	$\frac{3}{8}$	32.7	9.61	2.23	59.42	10.30	2.49	1.58
8×8	$\frac{1}{2}$	35.8	10.53	2.25	64.64	11.25	2.48	1.58
8×8	$\frac{3}{4}$	38.9	11.44	2.28	69.74	12.18	2.47	1.57
8×8	1	42.0	12.34	2.30	74.71	13.11	2.46	1.57
8×8	$\frac{1}{8}$	45.0	13.23	2.32	79.58	14.01	2.45	1.57
8×8	$\frac{1}{4}$	48.1	14.12	2.34	84.33	14.91	2.44	1.56
8×8	1	51.0	15.00	2.37	88.98	15.80	2.44	1.56
8×8	$\frac{1}{8}$	54.0	15.87	2.39	93.53	16.67	2.43	1.56
8×8	$\frac{1}{4}$	56.9	16.73	2.41	97.97	17.53	2.42	1.55

TABLE X
PROPERTIES OF STANDARD ANGLES HAVING UNEQUAL LEGS



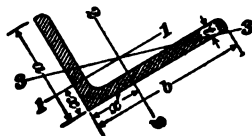
1	2	3	4	5	6	7	8	9	10	11	12	13
Widths Inches	Thickness Inch	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity From Back of Longer Flange, Inches	Moment of Inertia, Referred to the Inch Axis <i>I-I</i>	Section Modulus, Referred to the Inch Axis <i>I-I</i>	Radius of Gyration, Axis <i>I-I</i> Inches	Distance of Center of Gravity From Back of Shorter Flange, Inches	Moment of Inertia, Referred to the Inch Axis <i>2-2</i>	Section Modulus, Referred to the Inch Axis <i>2-2</i>	Radius of Gyration, Axis <i>2-2</i> Inches	Least Radius of Gyration, Axis <i>2-2</i> Inch
<i>b</i> × <i>a</i>	<i>t</i>		<i>A</i>	<i>x</i>	<i>I</i>	<i>Q</i>	<i>r</i>	<i>x'</i>	<i>I'</i>	<i>Q'</i>	<i>r'</i>	<i>r''</i>
2½ × 2	⅜	2.8	.81	.51	.29	.20	.60	.76	.51	.29	.79	.43
2½ × 2	½	3.6	1.06	.54	.37	.25	.59	.79	.65	.38	.78	.42
2½ × 2	⅝	4.5	1.31	.56	.45	.31	.58	.81	.79	.47	.78	.42
2½ × 2	¾	5.3	1.55	.58	.51	.36	.58	.83	.91	.55	.77	.42
2½ × 2	⅞	6.0	1.78	.60	.58	.41	.57	.85	1.03	.62	.76	.42
2½ × 2	1	6.8	2.00	.63	.64	.46	.56	.88	1.14	.70	.75	.42
3 × 2½	⅜	4.5	1.31	.66	.74	.40	.75	.91	1.17	.56	.95	.53
3 × 2½	½	5.5	1.62	.68	.90	.49	.74	.93	1.42	.69	.94	.53
3 × 2½	⅝	6.5	1.92	.71	1.04	.58	.74	.96	1.66	.81	.93	.52
3 × 2½	¾	7.5	2.21	.73	1.18	.66	.73	.98	1.88	.93	.92	.52
3 × 2½	⅞	8.5	2.50	.75	1.30	.74	.72	1.00	2.08	1.04	.91	.52
3 × 2½	1	9.4	2.78	.77	1.42	.82	.72	1.02	2.28	1.15	.91	.52
3½ × 2½	⅜	4.9	1.44	.61	.78	.41	.74	1.11	1.80	.75	1.12	.54
3½ × 2½	½	6.0	1.78	.64	.94	.50	.73	1.14	2.19	.93	1.11	.54
3½ × 2½	⅝	7.2	2.11	.66	1.09	.59	.72	1.16	2.56	1.09	1.10	.54
3½ × 2½	¾	8.3	2.43	.68	1.23	.68	.71	1.18	2.91	1.26	1.09	.54
3½ × 2½	⅞											
3½ × 2½	1	9.4	2.75	.70	1.36	.76	.70	1.20	3.24	1.41	1.09	.53
3½ × 2½	1 ⅛	10.4	3.06	.73	1.49	.84	.70	1.23	3.55	1.56	1.08	.53
3½ × 2½	1 ¼	11.4	3.36	.75	1.61	.92	.69	1.25	3.85	1.71	1.07	.53
3½ × 2½	1 ½	12.4	3.65	.77	1.72	.99	.69	1.27	4.13	1.85	1.06	.53
3½ × 3	⅜	6.6	1.93	.81	1.58	.72	.90	1.06	2.33	.95	1.10	.63
3½ × 3	½	7.8	2.30	.83	1.85	.85	.90	1.08	2.72	1.13	1.09	.62
3½ × 3	⅝	9.0	2.65	.85	2.09	.98	.89	1.10	3.10	1.29	1.08	.62
3½ × 3	¾	10.2	3.00	.88	2.33	1.10	.88	1.13	3.45	1.45	1.07	.62
3½ × 3	⅞	11.4	3.34	.90	2.55	1.21	.87	1.15	3.79	1.61	1.07	.62
3½ × 3	1											
3½ × 3	1 ⅛	12.5	3.67	.92	2.76	1.33	.87	1.17	4.11	1.76	1.06	.62
3½ × 3	1 ¼	13.6	4.00	.94	2.96	1.44	.86	1.19	4.41	1.91	1.05	.62
3½ × 3	1 ½	14.7	4.31	.96	3.15	1.54	.85	1.21	4.70	2.05	1.04	.62
3½ × 3	1 ¾	15.7	4.62	.98	3.33	1.65	.85	1.23	4.98	2.20	1.04	.62
4 × 3	1 ½	7.1	2.09	.70	1.05	.73	.89	1.20	3.38	1.23	1.27	.65

TABLE X—(Continued)
PROPERTIES OF STANDARD ANGLES HAVING UNEQUAL LEGS



1	2	3	4	5	6	7	8	9	10	11	12	13
Widths Inches	Thickness Inch	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity From Back of Longer Flange, Inches	Moment of Inertia, Referred to the Inch Axis 1-1	Section Modulus, Referred to the Inch Axis 1-1	Radius of Gyration, Axis 1-1 Inches	Distance of Center of Gravity From Back of Shorter Flange, Inches	Moment of Inertia, Referred to the Inch Axis 2-2	Section Modulus, Referred to the Inch Axis 2-2	Radius of Gyration Axis 2-2 Inches	Least Radius of Gyration, Axis 2-2 Inches
$b \times a$	t		A	x	I	Q	r	x'	I'	Q'	r'	r''
4×3	$\frac{3}{16}$	8.4	2.48	.78	1.92	.87	.88	1.28	3.96	1.46	1.26	.64
4×3	$\frac{1}{8}$	9.8	2.87	.80	2.18	.99	.87	1.30	4.52	1.68	1.25	.64
4×3	$\frac{3}{8}$	11.1	3.25	.83	2.42	1.12	.86	1.33	5.05	1.80	1.25	.64
4×3	$\frac{1}{2}$	12.3	3.62	.85	2.66	1.23	.86	1.35	5.55	2.09	1.24	.64
4×3	$\frac{5}{8}$	13.6	3.98	.87	2.87	1.35	.85	1.37	6.03	2.30	1.23	.64
4×3	$\frac{3}{4}$	14.8	4.34	.89	3.08	1.46	.84	1.39	6.49	2.49	1.22	.64
4×3	$\frac{7}{8}$	15.9	4.60	.92	3.28	1.57	.84	1.42	6.93	2.68	1.22	.64
4×3	1	17.1	5.03	.94	3.47	1.68	.83	1.44	7.35	2.87	1.21	.64
5×3	$\frac{3}{16}$	8.2	2.40	.68	1.75	.75	.85	1.68	6.26	1.80	1.61	.69
5×3	$\frac{1}{8}$	9.7	2.80	.70	2.04	.80	.84	1.70	7.37	2.24	1.61	.65
5×3	$\frac{3}{8}$	11.3	3.31	.73	2.32	1.02	.84	1.73	8.43	2.58	1.60	.65
5×3	$\frac{1}{2}$	12.8	3.75	.75	2.58	1.15	.83	1.75	9.45	2.91	1.59	.65
5×3	$\frac{3}{4}$	14.2	4.18	.77	2.83	1.27	.82	1.77	10.43	3.23	1.58	.65
5×3	$\frac{7}{8}$	15.7	4.61	.80	3.06	1.39	.82	1.80	11.37	3.55	1.57	.64
5×3	1	17.1	5.03	.82	3.29	1.51	.81	1.82	12.28	3.80	1.56	.64
5×3	$\frac{1}{2}$	18.5	5.44	.84	3.51	1.62	.80	1.84	13.15	4.16	1.55	.64
5×3	$\frac{3}{4}$	19.9	5.84	.86	3.71	1.74	.80	1.86	13.98	4.46	1.55	.64
5×3½	$\frac{3}{16}$	8.7	2.50	.84	2.72	1.02	1.03	1.59	6.60	1.94	1.61	.77
5×3½	$\frac{1}{8}$	10.4	3.05	.86	3.18	1.21	1.02	1.61	7.78	2.20	1.60	.70
5×3½	$\frac{3}{8}$	12.0	3.53	.88	3.63	1.39	1.01	1.63	8.90	2.64	1.59	.70
5×3½	$\frac{1}{2}$	13.6	4.00	.91	4.05	1.56	1.01	1.66	9.99	2.90	1.58	.75
5×3½	$\frac{3}{4}$	15.2	4.40	.93	4.45	1.73	1.00	1.68	11.03	3.32	1.57	.75
5×3½	1	16.7	4.92	.95	4.83	1.90	.99	1.70	12.03	3.65	1.56	.75
5×3½	$\frac{1}{2}$	18.3	5.37	.97	5.20	2.06	.98	1.72	12.99	3.97	1.56	.75
5×3½	$\frac{3}{4}$	19.8	5.81	1.00	5.55	2.22	.98	1.75	13.92	4.28	1.55	.75
5×3½	1	21.2	6.25	1.02	5.89	2.37	.97	1.77	14.81	4.58	1.54	.75
5×3½	$\frac{1}{2}$	22.7	6.67	1.04	6.21	2.52	.96	1.79	15.67	4.88	1.53	.75
6×3½	$\frac{1}{2}$	11.6	3.42	.70	3.34	1.23	.99	2.04	12.86	3.24	1.94	.77
6×3½	$\frac{3}{4}$	13.5	3.96	.81	3.81	1.41	.98	2.06	14.76	3.75	1.93	.70

TABLE X—(Continued)
PROPERTIES OF STANDARD ANGLES HAVING UNEQUAL LEGS



1	2	3	4	5	6	7	8	9	10	11	12	13
Widths Inches	Thickness Inch	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity From Back of Longer Flange, Inches	Moment of Inertia, Referred to the Inch Axis 1-1	Section Modulus, Referred to the Inch Axis 1-1	Radius of Gyration, Axis 1-1 Inches	Distance of Center of Gravity From Back of Shorter Flange, Inches	Moment of Inertia, Referred to the Inch Axis 2-2	Section Modulus, Referred to the Inch Axis 2-2	Radius of Gyration, Axis 2-2 Inches	Least Radius of Gyration, Axis 2-2 Inch
$b \times a$	t		A	x	I	Q	r	x'	I'	Q'	r'	r''
$6 \times 3\frac{1}{2}$	$\frac{1}{8}$	15.3	4.50	.83	4.25	1.59	.97	2.08	16.59	4.24	1.92	.76
$6 \times 3\frac{1}{2}$	$\frac{1}{16}$	17.1	5.03	.86	4.67	1.77	.96	2.11	18.37	4.72	1.91	.75
$6 \times 3\frac{1}{2}$	$\frac{3}{16}$	18.9	5.55	.88	5.08	1.94	.96	2.13	20.08	5.19	1.90	.75
$6 \times 3\frac{1}{2}$	$\frac{1}{4}$	20.6	6.06	.90	5.47	2.11	.95	2.15	21.74	5.65	1.89	.75
$6 \times 3\frac{1}{2}$	$\frac{5}{16}$	22.3	6.56	.93	5.84	2.27	.94	2.18	23.34	6.10	1.89	.75
$6 \times 3\frac{1}{2}$	$\frac{3}{8}$	24.0	7.06	.95	6.20	2.43	.94	2.20	24.89	6.55	1.88	.75
$6 \times 3\frac{1}{2}$	$\frac{7}{16}$	25.7	7.55	.97	6.55	2.59	.93	2.22	26.39	6.98	1.87	.75
$6 \times 3\frac{1}{2}$	$\frac{1}{2}$	27.3	8.03	.99	6.88	2.74	.93	2.24	27.84	7.41	1.86	.74
$6 \times 3\frac{1}{2}$	1	28.9	8.50	1.01	7.21	2.90	.92	2.26	29.24	7.83	1.85	.74
6×4	$\frac{3}{16}$	12.3	3.61	.94	4.90	1.60	1.17	1.94	13.47	3.32	1.93	.88
6×4	$\frac{1}{4}$	14.2	4.18	.96	5.60	1.85	1.16	1.96	15.46	3.83	1.92	.87
6×4	$\frac{5}{16}$	16.2	4.75	.99	6.27	2.08	1.15	1.99	17.40	4.33	1.91	.87
6×4	$\frac{3}{8}$	18.1	5.31	1.01	6.91	2.31	1.14	2.01	19.26	4.83	1.90	.87
6×4	$\frac{7}{16}$	19.9	5.86	1.03	7.52	2.54	1.13	2.03	21.07	5.31	1.90	.86
6×4	$\frac{1}{2}$	21.8	6.40	1.06	8.11	2.76	1.13	2.06	22.82	5.78	1.89	.86
6×4	$\frac{5}{8}$	23.6	6.94	1.08	8.68	2.97	1.12	2.08	24.51	6.25	1.88	.86
6×4	$\frac{3}{4}$	25.4	7.46	1.10	9.23	3.18	1.11	2.10	26.15	6.70	1.87	.86
6×4	1	27.2	7.98	1.12	9.75	3.39	1.11	2.12	27.73	7.15	1.86	.86
6×4	$\frac{1}{8}$	28.9	8.50	1.14	10.26	3.59	1.10	2.14	29.26	7.59	1.86	.85
6×4	1	30.6	9.00	1.17	10.75	3.79	1.09	2.17	30.75	8.02	1.85	.85
$7 \times 3\frac{1}{2}$	$\frac{1}{8}$	17.0	5.00	.78	4.41	1.62	.94	2.53	25.41	5.68	2.25	.89
$7 \times 3\frac{1}{2}$	$\frac{1}{16}$	19.1	5.59	.80	4.86	1.80	.93	2.55	28.18	6.33	2.25	.89
$7 \times 3\frac{1}{2}$	$\frac{3}{16}$	21.0	6.17	.82	5.28	1.97	.93	2.57	30.86	6.97	2.24	.89
$7 \times 3\frac{1}{2}$	$\frac{1}{4}$	23.0	6.75	.85	5.69	2.14	.92	2.60	33.47	7.60	2.23	.89
$7 \times 3\frac{1}{2}$	$\frac{5}{16}$	24.9	7.31	.87	6.08	2.31	.91	2.62	35.99	8.22	2.22	.88
$7 \times 3\frac{1}{2}$	$\frac{3}{8}$	26.8	7.87	.89	6.46	2.48	.91	2.64	38.45	8.82	2.21	.88
$7 \times 3\frac{1}{2}$	$\frac{7}{16}$	28.7	8.42	.92	6.83	2.64	.90	2.67	40.82	9.42	2.20	.88
$7 \times 3\frac{1}{2}$	$\frac{1}{2}$	30.5	8.97	.94	7.18	2.80	.89	2.69	43.13	10.00	2.19	.88
$7 \times 3\frac{1}{2}$	1	32.3	9.50	.96	7.53	2.96	.89	2.71	45.37	10.58	2.19	.88

TABLE XI

THICKNESSES OF ANGLES FOR WHICH NOMINAL AND ACTUAL WIDTHS ARE EQUAL

Equal Legs Inches	Thicknesses Inch	Unequal Legs Inches	Thicknesses Inch
8×8	$\frac{1}{2}$ and $\frac{3}{4}$	7×3½	$\frac{1}{2}$ and $\frac{3}{4}$
6×6	$\frac{3}{8}$ and $\frac{1}{2}$	6×4	$\frac{3}{8}$ and $\frac{9}{16}$
4×4	$\frac{5}{16}$ and $\frac{1}{2}$	6×3½	$\frac{3}{8}$ and $\frac{9}{16}$
3½×3½	$\frac{5}{16}$ and $\frac{1}{2}$	5×3½	$\frac{5}{16}$ and $\frac{1}{2}$
3×3	$\frac{1}{2}$	5×3	$\frac{5}{16}$ and $\frac{1}{2}$
2½×2½	$\frac{3}{16}$	4×3	$\frac{5}{16}$ and $\frac{1}{2}$
2×2	$\frac{3}{16}$	3½×3	$\frac{5}{16}$
1½×1½	$\frac{3}{16}$	3½×2½	$\frac{1}{2}$
1½×1½	$\frac{1}{8}$	3×2½	$\frac{1}{2}$
1½×1½	$\frac{1}{8}$	2½×2	$\frac{3}{16}$

TABLE XII

STANDARD GAUGE LINES FOR ANGLES, IN INCHES




						
L	G	Maximum Rivet	L	G ₁	G ₂	Maximum Rivet
8	4½	$\frac{7}{8}$	8	3	3	$\frac{7}{8}$
7	4	$\frac{7}{8}$	7	2½	3	$\frac{7}{8}$
6	3½	$\frac{7}{8}$	6	2¼	2¼	$\frac{7}{8}$
5	3	$\frac{7}{8}$	5	2	1¾	$\frac{7}{8}$
4	2¼	$\frac{7}{8}$	For 8-Inch Leg Only 			
3½	2	$\frac{7}{8}$				
3	1¾	$\frac{7}{8}$				
2½	1¾	$\frac{3}{4}$				
2	1½	$\frac{5}{8}$				
1¾	1	$\frac{1}{2}$				
1½	$\frac{7}{8}$	$\frac{3}{8}$				
1½	$\frac{3}{4}$	$\frac{3}{8}$				

TABLE XIII

PROPERTIES OF STANDARD CHANNELS

TABLE XIII

PROPERTIES OF STANDARD CHANNELS

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17											
Depth of Channel Inches	d	Weight per Foot Pounds	Area of Section Square Inches	Thickness of Web Inch	Width of Flange Inches	Moment of Inertia, Referred to the Inch Axis 1-1	I	Section Modulus, Referred to the Inch Axis 1-1	Q	Radius of Gyration, Axis 1-1 Inches	r	Moment of Inertia, Referred to the Inch Axis 2-2	I'	Section Modulus, Referred to the Inch Axis 2-2	Q'	Radius of Gyration, Axis 2-2 Inches	Distance of Center of Gravity from Outside of Web Inches	x	Clear Distance Between Fillets Inches	c	Gauge of Flanges Inches	b	Grip of Flange Rivets Inches	G	Maximum Flange Rivet or Bolt Inches	Distance Between Backs of C's to Gyration Equal	D
3	4.00	1.19	.17	1.41	1.6	1.1	1.17	.20	.21	.41	.44	.21	.20	.29	.32	.45	.46	.46	.21	.21	1	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	2.06	$\frac{11}{16}$
3	5.00	1.47	.26	1.50	1.8	1.2	1.12	.25	.24	.41	.44	.24	.25	.32	.32	.45	.46	.46	.24	.24	1	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	1.96	$\frac{11}{16}$
3	6.00	1.76	.36	1.60	2.1	1.4	1.08	.31	.27	.42	.46	.31	.31	.35	.35	.46	.46	.46	.24	.24	1	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	1.85	$\frac{11}{16}$
4	5.25	1.55	.18	1.58	3.8	1.9	1.56	.32	.29	.45	.46	.32	.32	.38	.38	.50	.49	.49	.21	.21	1	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	2.79	$\frac{11}{16}$
4	6.25	1.84	.25	1.65	4.2	2.1	1.51	.38	.32	.45	.46	.32	.38	.45	.45	.49	.48	.48	.24	.24	1	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	2.56	$\frac{11}{16}$
4	7.25	2.13	.33	1.73	4.6	2.3	1.46	.44	.35	.46	.46	.35	.44	.46	.46	.49	.51	.51	.24	.24	1	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	2.34	$\frac{11}{16}$
5	6.50	1.95	.19	1.75	7.4	3.0	1.95	.48	.38	.50	.49	.48	.48	.54	.54	.49	.49	.49	.34	.34	1	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$		
5	9.00	2.65	.33	1.89	8.9	3.5	1.83	.64	.45	.49	.48	.64	.64	.45	.45	.49	.48	.48	.34	.34	1	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$		
5	11.50	3.38	.48	2.04	10.4	4.2	1.75	.82	.54	.49	.51	.82	.82	.54	.54	.49	.51	.51	.34	.34	1	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$	$\frac{11}{16}$		

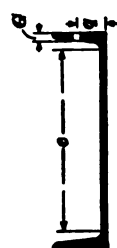


TABLE XIII—(Continued)
PROPERTIES OF STANDARD CHANNELS



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Depth of Channel Inches	Weight per Foot Pounds	Area of Section Square Inches	Thickness of Web Inch	Width of Flange Inches	Moment of Inertia, Referred to the Inch Axis 1-1	Section Modulus, Referred to the Inch Axis 1-1	Radius of Gyration Axis 1-1 Inches	Moment of Inertia Referred to the Inch Axis 2-2	Section Modulus, Referred to the Inch Axis 2-2	Radius of Gyration, Axis 2-2 Inch	Distance of Center of Gravity From Outside of Web Inch	Clear Distance Between Pillets Inches	Gauge of Flanges Inches	Grip of Flange Rivets Inch	Maximum Flange Rivet or Bolt Inch	Distance Between Backs of C's to Make Radii of Gyration Equal
6	8.00	2.38	.20	1.92	13.0	4.3	2.34	.70	.50	.54	.52	4½	1½	1½	1½	3.52
6	10.50	3.00	.32	2.04	15.1	5.0	2.21	.88	.57	.53	.50	4½	1½	1½	1½	3.28
6	13.00	3.82	.44	2.16	17.3	5.8	2.13	1.07	.65	.53	.52	4½	1½	1½	1½	3.09
6	15.50	4.56	.56	2.28	19.5	6.5	2.07	1.28	.74	.53	.55	4½	1½	1½	1½	2.91
7	9.75	2.85	.21	2.09	21.1	6.0	2.72	.98	.63	.59	.55	5½	1½	1½	1½	4.22
7	12.25	3.60	.32	2.20	24.2	6.9	2.59	1.19	.71	.57	.53	5½	1½	1½	1½	3.99
7	14.75	4.34	.42	2.30	27.2	7.8	2.50	1.40	.79	.57	.53	5½	1½	1½	1½	3.80
7	17.25	5.07	.53	2.41	30.2	8.6	2.44	1.62	.87	.56	.55	5½	1½	1½	1½	3.64
7	19.75	5.81	.63	2.51	33.2	9.5	2.39	1.85	.96	.56	.58	5½	1½	1½	1½	3.48

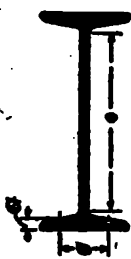
BRIDGE TABLES

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8	11.25	3.35	.22	2.26	32.3	8.1	3.10	1.33	.79	.63	.58	6†	1†	†	†	4.94
8	13.75	4.04	.31	2.35	36.0	9.0	2.98	1.55	.87	.62	.56	6†	1†	†	†	4.72
8	16.25	4.78	.40	2.44	39.9	10.0	2.89	1.78	.95	.61	.56	6†	1†	†	†	4.54
8	18.75	5.51	.49	2.53	43.8	11.0	2.82	2.01	1.02	.60	.57	6†	1†	†	†	4.38
8	21.25	6.25	.58	2.62	47.8	11.9	2.76	2.25	1.11	.60	.59	6†	1†	†	†	4.23
9	13.25	3.89	.23	2.43	47.3	10.5	3.49	1.77	.97	.67	.61	7†	1†	†	†	5.63
9	15.00	4.41	.29	2.49	50.9	11.3	3.40	1.95	1.03	.66	.59	7†	1†	†	†	5.49
9	20.00	5.88	.45	2.65	60.8	13.5	3.21	2.45	1.19	.65	.58	7†	1†	†	†	5.12
9	25.00	7.35	.61	2.81	70.7	15.7	3.10	2.98	1.36	.64	.62	7†	1†	†	†	4.84
10	15.00	4.46	.24	2.60	66.9	13.4	3.87	2.30	1.17	.72	.64	8†	1†	†	†	6.33
10	20.00	5.88	.38	2.74	78.7	15.7	3.66	2.85	1.34	.70	.61	8†	1†	†	†	5.97
10	25.00	7.35	.53	2.89	91.0	18.2	3.52	3.40	1.50	.68	.62	8†	2	†	†	5.67
10	30.00	8.82	.68	3.04	103.2	20.6	3.42	3.99	1.67	.67	.65	8†	2	†	†	5.40
10	35.00	10.29	.82	3.18	115.5	23.1	3.35	4.66	1.87	.67	.69	8†	2	†	†	5.17
12	20.50	6.03	.28	2.94	128.1	21.4	4.61	3.91	1.75	.81	.70	10	1†	†	†	7.67
12	25.00	7.35	.39	3.05	144.0	24.0	4.43	4.53	1.91	.78	.68	10	1†	†	†	7.36
12	30.00	8.82	.51	3.17	161.6	26.9	4.28	5.21	2.09	.77	.68	10	2	†	†	7.07
12	35.00	10.29	.64	3.30	179.3	29.9	4.17	5.90	2.27	.76	.69	10	2	†	†	6.81
12	40.00	11.76	.76	3.42	196.9	32.8	4.09	6.63	2.46	.75	.72	10	2	†	†	6.60
15	33.00	9.90	.40	3.40	312.6	41.7	5.62	8.23	3.16	.91	.79	12†	1†	†	†	9.50
15	35.00	10.29	.43	3.43	319.9	42.7	5.57	8.48	3.22	.91	.79	12†	1†	†	†	9.43
15	40.00	11.76	.52	3.52	347.5	46.3	5.44	9.39	3.43	.89	.78	12†	1†	†	†	9.15
15	45.00	13.24	.62	3.62	375.1	50.0	5.32	10.29	3.63	.88	.79	12†	2†	†	†	8.92
15	50.00	14.71	.72	3.72	402.7	53.7	5.23	11.22	3.85	.87	.80	12†	2†	†	†	8.71
15	55.00	16.18	.82	3.82	430.2	57.4	5.16	12.19	4.07	.87	.82	12†	2†	†	†	8.53



TABLE XIV
PROPERTIES OF STANDARD I BEAMS



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Depth of Beam Inches	Weight per Foot Pounds	Area of Section Square Inches	Thickness of Web Inches	Width of Flange Inches	Moment of Inertia, Axis 1-1 Referred to the Inch	Section Modulus, Axis 1-1 Referred to the Inch	Radius of Gyration Axis 1-1 Inches	Moment of Inertia, Axis 2-2 Referred to the Inch	Radius of Gyration Axis 2-2 Inches	Clear Distance Between Fillets Inches	Gauge of Flanges Inches	Grip of Flange Rivets Inches	Maximum Flange Rivet or Bolt Inch	Distance Between Centers of Webs to Make Radii of Gy- ration Equal
3	5.50	1.63	.17	2.33	2.5	1.7	1.23	.46	.53	1.1	1.1	1.1		
3	6.50	1.91	.26	2.42	2.7	1.8	1.19	.53	.52	1.1	1.1	1.1		
3	7.50	2.21	.36	2.52	2.9	1.9	1.15	.60	.52	1.1	1.1	1.1		
4	7.50	2.21	.19	2.66	6.0	3.0	1.64	.77	.59	2.1	1.1	1.1		
4	8.50	2.50	.26	2.73	6.4	3.2	1.59	.85	.58	2.1	1.1	1.1		
4	9.50	2.79	.34	2.81	6.7	3.4	1.55	.93	.58	2.1	1.1	1.1		
4	10.50	3.09	.41	2.88	7.1	3.6	1.52	1.01	.57	2.1	1.1	1.1		
5	9.75	2.87	.21	3.00	12.1	4.8	2.05	1.23	.65	3.1	1.1	1.1		
5	12.25	3.60	.36	3.15	13.6	5.4	1.94	1.45	.63	3.1	1.1	1.1		
5	14.75	4.34	.50	3.29	15.2	6.1	1.87	1.70	.63	3.1	1.1	1.1		
6	12.25	3.61	.23	3.33	21.8	7.3	2.46	1.85	.72	4.1	2	1.1		
6	14.75	4.34	.35	3.45	24.0	8.0	2.35	2.09	.68	4.1	2	1.1		
6	17.25	5.07	.47	3.57	26.2	8.7	2.27	2.36	.68	4.1	2	1.1		
7	15.00	4.42	.25	3.66	36.2	10.4	2.86	2.67	.78	5.1	2.1	1.1		
7	17.50	5.15	.35	3.76	39.2	11.2	2.76	2.94	.76	5.1	2.1	1.1		
7	20.00	5.88	.46	3.87	42.2	12.1	2.68	3.24	.74	5.1	2.1	1.1		



4.70
4.49
4.33
5.50
5.31
5.15

8	18.00	5.33	.27	4.00	56.9	14.2	3.27	3.78	.84	6 ¹	2 ¹	11 ¹	6.32
8	20.50	6.03	.36	4.09	60.6	15.1	3.17	4.07	.82	6 ¹	2 ¹	11 ¹	6.12
8	23.00	6.76	.45	4.18	64.5	16.1	3.09	4.39	.81	6 ¹	2 ¹	11 ¹	5.96
8	25.50	7.50	.54	4.27	68.4	17.1	3.02	4.75	.80	6 ¹	2 ¹	11 ¹	5.82
9	21.00	6.31	.29	4.33	84.9	18.9	3.67	5.16	.90	7	2 ¹	16 ¹	7.12
9	25.00	7.35	.41	4.45	91.9	20.4	3.54	5.65	.88	7	2 ¹	16 ¹	6.86
9	30.00	8.82	.57	4.61	101.9	22.6	3.40	6.42	.85	7	2 ¹	16 ¹	6.58
9	35.00	10.29	.73	4.77	111.8	24.8	3.29	7.31	.84	7	2 ¹	16 ¹	6.36
10	25.00	7.37	.31	4.66	122.1	24.4	4.07	6.89	.97	8	2 ¹	17 ¹	7.91
10	30.00	8.82	.45	4.80	134.2	26.8	3.90	7.65	.93	8	2 ¹	17 ¹	7.57
10	35.00	10.29	.60	4.95	146.4	29.3	3.77	8.52	.91	8	2 ¹	17 ¹	7.32
10	40.00	11.76	.75	5.10	158.7	31.7	3.67	9.50	.90	8	2 ¹	17 ¹	7.12
12	31.50	9.26	.35	5.00	215.8	36.0	4.83	9.50	1.01	9 ¹	2 ¹	17 ¹	9.45
12	35.00	10.29	.44	5.09	228.3	38.0	4.71	10.07	.99	9 ¹	2 ¹	17 ¹	9.21
12	40.00	11.84	.46	5.25	268.9	44.8	4.77	13.81	1.08	9 ¹	3	17 ¹	9.29
12	45.00	13.24	.58	5.37	285.7	47.6	4.65	14.89	1.06	9 ¹	3	17 ¹	9.06
12	50.00	14.71	.70	5.49	303.3	50.6	4.54	16.12	1.05	9 ¹	3	17 ¹	8.83
12	55.00	16.18	.82	5.61	321.0	53.5	4.45	17.46	1.04	9 ¹	3	17 ¹	8.65
15	42.00	12.48	.41	5.50	441.7	58.9	5.95	14.62	1.08	12 ¹	3	18 ¹	11.70
15	45.00	13.24	.46	5.55	455.8	60.8	5.87	15.00	1.07	12 ¹	3	18 ¹	11.54
15	50.00	14.71	.56	5.65	483.4	64.5	5.73	16.04	1.04	12 ¹	3	18 ¹	11.27
15	55.00	16.18	.66	5.75	511.0	68.1	5.62	17.06	1.02	12 ¹	3	18 ¹	11.05
15	60.00	17.67	.59	6.00	609.0	81.2	5.87	25.96	1.21	11 ¹	3 ¹	18 ¹	11.45
15	65.00	19.12	.69	6.10	636.0	84.8	5.77	27.42	1.20	11 ¹	3 ¹	18 ¹	11.29
15	70.00	20.59	.78	6.19	663.6	88.5	5.68	29.00	1.19	11 ¹	3 ¹	18 ¹	11.11
15	75.00	22.06	.88	6.29	691.2	92.2	5.60	30.68	1.18	11 ¹	3 ¹	18 ¹	10.95
15	80.00	23.81	.81	6.40	795.5	106.1	5.78	41.76	1.32	11	3 ¹	18 ¹	11.25
15	85.00	25.00	.89	6.48	817.8	109.0	5.72	43.57	1.32	11	3 ¹	18 ¹	11.13
15	90.00	26.47	.99	6.58	845.4	112.7	5.65	45.91	1.32	11	3 ¹	18 ¹	10.99
15	95.00	27.94	1.08	6.67	872.9	116.4	5.59	48.37	1.32	11	3 ¹	18 ¹	10.86
15	100.00	29.41	1.18	6.77	900.5	120.1	5.53	50.98	1.31	11	3 ¹	18 ¹	10.75
18	55.00	15.93	.46	6.00	795.6	88.4	7.07	21.19	1.15	15 ¹	3 ¹	18 ¹	13.95
18	60.00	17.05	.56	6.10	841.8	93.5	6.91	22.38	1.13	15 ¹	3 ¹	18 ¹	13.65
18	65.00	19.12	.64	6.18	881.5	97.9	6.79	23.47	1.11	15 ¹	3 ¹	18 ¹	13.40
18	70.00	20.59	.72	6.26	921.3	102.4	6.69	24.62	1.09	15 ¹	3 ¹	18 ¹	13.20

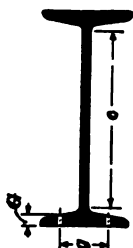
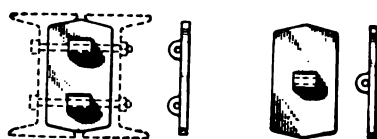


TABLE XIV—(Continued)
PROPERTIES OF STANDARD I BEAMS



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Depth of Beam Inches	Weight per Foot Pounds	Area of Section Square Inches	Thickness of Web Inches	Width of Flange Inches	Moment of Inertia, Axis 1-1	Section Modulus, Axis 1-1	Radius of Gyration Axis 1-1 Inches	Moment of Inertia, Axis 2-2 Inches	Radius of Gyration Axis 2-2 Inches	Clear Distance Between Flanges Inches	Gauge of Flanges Inches	Grip of Flange Rivets Inches	Maximum Flange Rivet or Bolt Inch	Distance Between Centers of Webs to Make Radii of Gy- ration Equal
20	65.00	19.08	.50	6.25	1,169.6	117.0	7.83	27.86	1.21	17	3 1/4			15.47
20	70.00	20.59	.58	6.33	1,210.9	122.0	7.70	29.04	1.19	17	3 1/4			15.21
20	75.00	22.00	.65	6.40	1,268.9	126.9	7.58	30.25	1.17	17	3 1/4			14.98
20	80.00	23.73	.60	7.00	1,466.5	146.7	7.86	45.81	1.39	16 1/2	4			15.47
20	85.00	25.00	.66	7.00	1,508.7	150.9	7.77	47.25	1.37	16 1/2	4			15.30
20	90.00	26.47	.74	7.14	1,557.8	155.8	7.67	48.98	1.36	16 1/2	4			15.10
20	95.00	27.94	.81	7.21	1,606.8	160.7	7.58	50.78	1.35	16 1/2	4			14.92
20	100.00	29.41	.88	7.28	1,655.8	165.6	7.50	52.65	1.34	16 1/2	4			14.76
24	80.00	23.32	.50	7.00	2,087.9	174.0	9.46	42.86	1.36	20 1/2	4			18.72
24	85.00	25.00	.57	7.07	2,108.6	180.7	9.31	44.35	1.33	20 1/2	4			18.43
24	90.00	26.47	.63	7.13	2,239.1	186.6	9.20	45.70	1.31	20 1/2	4			18.21
24	95.00	27.94	.69	7.19	2,300.6	192.5	9.09	47.10	1.30	20 1/2	4			17.99
24	100.00	29.41	.75	7.25	2,380.3	198.4	9.00	48.56	1.28	20 1/2	4			17.82

TABLE XV
CAST-IRON SEPARATORS



Depth of Beam Inches	Weight of Beam Pounds per Foot	Center to Center of Beams Inches	Center to Center of Bolts Inches	Thickness of Separator Inch	Weight of Separator Pounds
6	12.25	4	*	$\frac{1}{2}$	4
7	15.00	$4\frac{1}{4}$	*	$\frac{1}{2}$	4
8	18.00	$4\frac{1}{2}$	*	$\frac{1}{2}$	6
9	21.00	5	*	$\frac{1}{2}$	7
10	25.00	$5\frac{1}{2}$	*	$\frac{1}{2}$	8
12	31.50	$5\frac{3}{4}$	*	$\frac{1}{2}$	10
12	40.00	6	*	$\frac{1}{2}$	10
12	31.50	$5\frac{3}{4}$	5	$\frac{1}{2}$	11
12	40.00	6	5	$\frac{1}{2}$	11
15	42.00	$6\frac{1}{4}$	$7\frac{1}{2}$	$\frac{1}{2}$	15
15	60.00	$6\frac{3}{4}$	$7\frac{1}{2}$	$\frac{1}{2}$	15
15	80.00	$7\frac{1}{4}$	$7\frac{1}{2}$	$\frac{1}{2}$	15
18	55.00	$6\frac{3}{4}$	9	$\frac{5}{8}$	16
20	65.00	7	12	$\frac{5}{8}$	25
20	80.00	$7\frac{1}{4}$	12	$\frac{5}{8}$	28
24	80.00	$7\frac{1}{4}$	12	$\frac{5}{8}$	32

* For these beams, but one bolt is used.

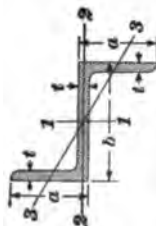


TABLE XVI
PROPERTIES OF Z BARS

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Depth of Bar Inches	Length of Legs Inches	Thickness of Web and Legs Inch	Weight per Foot Pounds	Area of Section Square Inches	Moment of Inertia, Referred to the Inch Axis 1-1	Section Modulus, Referred to the Inch Axis 1-1	Radius of Gyration, Axis 1-1 Inches	Moment of Inertia, Referred to the Inch Axis 2-2	Section Modulus, Referred to the Inch Axis 2-2	Radius of Gyration, Axis 2-2 Inches	Least Radius of Gyration, Axis 3-3 Inch	Gauge in Web Inches	Gauge in Leg Inches
3	2 1/16	1/4	6.7	1.97	2.87	1.92	1.21	2.81	1.10	1.19	.55	1 1/8	1 1/8
3 1/8	2 1/8	1/4	8.4	2.48	3.64	2.38	1.21	3.64	1.40	1.21	.55	1 1/8	1 1/8
3	2 1/16	3/8	9.7	2.86	3.85	2.57	1.16	3.92	1.57	1.17	.54	1 1/8	1 1/8
3 1/8	2 1/8	1/2	11.4	3.36	4.57	2.98	1.17	4.75	1.88	1.19	.55	1 1/8	1 1/8
3	2 1/16	1/2	12.5	3.69	4.59	3.06	1.12	4.85	1.99	1.15	.53	1 1/8	1 1/8
3 1/8	2 1/8	1/2	14.2	4.18	5.26	3.43	1.12	5.68	2.30	1.17	.54	1 1/8	1 1/8
4	3 1/16	1/2	8.2	2.41	6.28	3.14	1.62	4.23	1.44	1.33	.67	2	2
4 1/8	3 1/8	1/2	10.3	3.03	7.94	3.91	1.62	5.46	1.84	1.34	.68	2	2
4 1/8	3 1/8	3/4	12.4	3.66	9.63	4.67	1.62	6.77	2.26	1.36	.69	2	2



4	$3\frac{1}{16}$	$1\frac{7}{8}$	13.8	4.05	9.66	4.83	1.54	6.73	2.37	1.29	.66	2
$4\frac{1}{8}$	$3\frac{1}{8}$	$\frac{1}{2}$	15.8	4.66	11.18	5.50	1.55	7.96	2.77	1.31	.67	2
$4\frac{1}{4}$	$3\frac{1}{4}$	$\frac{1}{4}$	17.9	5.27	12.74	6.18	1.55	9.26	3.19	1.32	.68	2
4	$3\frac{1}{8}$	$\frac{3}{8}$	18.9	5.55	12.11	6.05	1.48	8.73	3.18	1.25	.65	2
$4\frac{1}{8}$	$3\frac{1}{8}$	$\frac{1}{2}$	20.9	6.14	13.52	6.65	1.48	9.95	3.58	1.27	.67	2
$4\frac{1}{4}$	$3\frac{1}{4}$	$\frac{1}{4}$	23.0	6.75	14.97	7.26	1.49	11.24	4.00	1.29	.68	2
5	$3\frac{1}{4}$	$1\frac{5}{8}$	11.6	3.40	13.36	5.34	1.98	6.18	2.00	1.35	.75	$2\frac{1}{2}$
$5\frac{1}{8}$	$3\frac{1}{8}$	$\frac{3}{8}$	13.9	4.10	16.18	6.39	1.99	7.65	2.45	1.37	.76	$2\frac{1}{2}$
$5\frac{1}{4}$	$3\frac{1}{4}$	$\frac{1}{2}$	16.4	4.81	19.07	7.44	1.99	9.20	2.92	1.38	.76	$2\frac{1}{2}$
5	$3\frac{1}{4}$	$\frac{1}{2}$	17.9	5.25	19.19	7.68	1.91	9.05	3.02	1.31	.74	$2\frac{1}{2}$
$5\frac{1}{8}$	$3\frac{1}{8}$	$\frac{1}{4}$	20.2	5.94	21.83	8.62	1.92	10.51	3.47	1.33	.75	$2\frac{1}{2}$
$5\frac{1}{4}$	$3\frac{1}{4}$	$\frac{3}{8}$	22.6	6.64	24.53	9.57	1.92	12.06	3.94	1.35	.76	$2\frac{1}{2}$
5	$3\frac{1}{4}$	$\frac{1}{2}$	23.7	6.96	23.68	9.47	1.84	11.37	3.91	1.28	.73	$2\frac{1}{2}$
$5\frac{1}{8}$	$3\frac{1}{8}$	$\frac{1}{4}$	26.0	7.64	26.16	10.34	1.85	12.83	4.37	1.30	.74	$2\frac{1}{2}$
$5\frac{1}{4}$	$3\frac{1}{4}$	$\frac{3}{8}$	28.3	8.33	28.70	11.20	1.86	14.37	4.84	1.31	.76	$2\frac{1}{2}$
6	$3\frac{1}{4}$	$\frac{3}{8}$	15.6	4.59	25.32	8.44	2.35	9.11	2.75	1.41	.83	3
$6\frac{1}{8}$	$3\frac{1}{8}$	$\frac{1}{2}$	18.3	5.39	29.80	9.83	2.35	10.94	3.27	1.43	.83	3
$6\frac{1}{4}$	$3\frac{1}{4}$	$\frac{1}{4}$	21.0	6.19	34.36	11.22	2.36	12.87	3.81	1.44	.84	3
6	$3\frac{1}{4}$	$\frac{1}{2}$	22.7	6.68	34.64	11.55	2.28	12.59	3.91	1.37	.81	3
$6\frac{1}{8}$	$3\frac{1}{8}$	$\frac{3}{8}$	25.4	7.46	38.87	12.82	2.28	14.41	4.44	1.39	.82	3
$6\frac{1}{4}$	$3\frac{1}{4}$	$\frac{1}{4}$	28.1	8.25	43.18	14.10	2.29	16.34	4.98	1.41	.84	3
6	$3\frac{1}{4}$	$\frac{1}{2}$	29.3	8.63	42.12	14.04	2.21	15.44	4.91	1.34	.81	3
$6\frac{1}{8}$	$3\frac{1}{8}$	$\frac{1}{4}$	31.9	9.39	46.13	15.22	2.22	17.27	5.47	1.36	.82	3
$6\frac{1}{4}$	$3\frac{1}{4}$	$\frac{3}{8}$	34.6	10.17	50.22	16.40	2.22	19.18	6.02	1.37	.83	3

TABLE XVII
DECIMAL EQUIVALENTS

Fractional Parts of an Inch	Inches											Decimal Equivalents of an Inch
	Decimal Parts of a Foot											
	1	2	3	4	5	6	7	8	9	10	11	
$\frac{1}{16}$.0000	.0625	.1250	.1875	.2500	.3125	.3750	.4375	.5000	.5625	.6250	$\frac{1}{16}$
$\frac{1}{8}$.0026	.0650	.1275	.1900	.2525	.3150	.3775	.4400	.5025	.5650	.6275	$\frac{1}{8}$
$\frac{3}{16}$.0052	.0675	.1350	.2025	.2700	.3375	.4050	.4725	.5400	.6075	.6750	$\frac{3}{16}$
$\frac{1}{4}$.0078	.0701	.1402	.2103	.2804	.3505	.4206	.4907	.5608	.6309	.7010	$\frac{1}{4}$
$\frac{5}{16}$.0104	.0727	.1454	.2181	.2908	.3635	.4362	.5089	.5816	.6543	.7270	$\frac{5}{16}$
$\frac{3}{8}$.0130	.0753	.1506	.2259	.2991	.3744	.4497	.5250	.5991	.6744	.7497	$\frac{3}{8}$
$\frac{7}{16}$.0156	.0779	.1558	.2337	.3116	.3895	.4674	.5453	.6232	.7011	.7790	$\frac{7}{16}$
$\frac{1}{2}$.0182	.0805	.1610	.2415	.3220	.4025	.4830	.5635	.6440	.7245	.8050	$\frac{1}{2}$
$\frac{9}{16}$.0208	.0831	.1662	.2493	.3324	.4155	.4986	.5817	.6648	.7479	.8310	$\frac{9}{16}$
$\frac{5}{8}$.0234	.0857	.1714	.2571	.3428	.4285	.5142	.6000	.6857	.7714	.8571	$\frac{5}{8}$
$\frac{11}{16}$.0260	.0883	.1766	.2649	.3532	.4415	.5298	.6181	.7064	.7947	.8830	$\frac{11}{16}$
$\frac{3}{4}$.0286	.0909	.1818	.2727	.3636	.4545	.5454	.6363	.7272	.8181	.9090	$\frac{3}{4}$

BRIDGE TABLES

29

1	.0313	.1146	.1979	.2813	.3646	.4479	.5313	.6146	.6979	.7813	.8646	.9479	1	.3750
2	.0339	.1172	.2005	.2839	.3672	.4505	.5339	.6172	.7005	.7839	.8672	.9505	2	.4063
3	.0365	.1198	.2031	.2865	.3698	.4531	.5365	.6198	.7031	.7865	.8698	.9531	3	.4375
4	.0391	.1224	.2057	.2891	.3724	.4557	.5391	.6224	.7057	.7891	.8724	.9557	4	.4688
5	.0417	.1250	.2083	.2917	.3750	.4583	.5417	.6250	.7083	.7917	.8750	.9583	5	.5000
6	.0443	.1276	.2109	.2943	.3776	.4609	.5443	.6276	.7109	.7943	.8776	.9609	6	.5313
7	.0469	.1302	.2135	.2969	.3802	.4635	.5469	.6302	.7135	.7969	.8802	.9635	7	.5625
8	.0495	.1328	.2161	.2995	.3828	.4661	.5495	.6328	.7161	.7995	.8828	.9661	8	.5938
9	.0521	.1354	.2188	.3021	.3854	.4688	.5521	.6354	.7188	.8021	.8854	.9688	9	.6250
10	.0547	.1380	.2214	.3047	.3880	.4714	.5547	.6380	.7214	.8047	.8880	.9714	10	.6563
11	.0573	.1406	.2240	.3073	.3906	.4740	.5573	.6406	.7240	.8073	.8906	.9740	11	.6875
12	.0599	.1432	.2266	.3099	.3932	.4766	.5599	.6432	.7266	.8099	.8932	.9766	12	.7188
13	.0625	.1458	.2292	.3125	.3958	.4792	.5625	.6458	.7292	.8125	.8958	.9792	13	.7500
14	.0651	.1484	.2318	.3151	.3984	.4818	.5651	.6484	.7318	.8151	.8984	.9818	14	.7813
15	.0677	.1510	.2344	.3177	.4010	.4844	.5677	.6510	.7344	.8177	.9010	.9844	15	.8125
16	.0703	.1536	.2370	.3203	.4036	.4870	.5703	.6536	.7370	.8203	.9036	.9870	16	.8438
17	.0729	.1563	.2396	.3229	.4063	.4896	.5729	.6563	.7396	.8229	.9063	.9896	17	.8750
18	.0755	.1589	.2422	.3255	.4089	.4922	.5755	.6589	.7422	.8255	.9089	.9922	18	.9063
19	.0781	.1615	.2448	.3281	.4115	.4948	.5781	.6615	.7448	.8281	.9115	.9948	19	.9375
20	.0807	.1641	.2474	.3307	.4141	.4974	.5807	.6641	.7474	.8307	.9141	.9974	20	.9688

TABLE XVIII
PROPERTIES AND PRINCIPAL DIMENSIONS OF STANDARD
T RAILS



1	2	3	4	5	6	7	8	9
Weight per Yard Pounds	Area Square Inches	b Inches	d Inches	k Inches	t Inch	x Inches	Axis 1-1	
							Moment of Inertia I	Section Modulus Q
16	1.57	2 $\frac{1}{4}$	2 $\frac{1}{4}$	1 $\frac{3}{16}$	$\frac{3}{2}$	1.10	1.13	.99
20	2.00	2 $\frac{3}{8}$	2 $\frac{3}{8}$	1 $\frac{3}{8}$	$\frac{1}{2}$	1.2	1.5	1.2
25	2.5	2 $\frac{3}{4}$	2 $\frac{3}{4}$	1 $\frac{1}{2}$	$\frac{13}{16}$	1.4	2.4	1.7
30	2.9	3 $\frac{1}{16}$	3	1 $\frac{11}{16}$	$\frac{11}{32}$	1.5	3.7	2.4
35	3.4	3 $\frac{3}{8}$	3 $\frac{1}{16}$	1 $\frac{27}{32}$	$\frac{13}{32}$	1.5	4.3	2.8
40	3.9	3 $\frac{1}{2}$	3 $\frac{1}{2}$	1 $\frac{37}{32}$	$\frac{23}{64}$	1.7	6.0	3.4
45	4.4	3 $\frac{3}{4}$	3 $\frac{1}{2}$	1 $\frac{59}{64}$	$\frac{3}{8}$	1.8	7.6	3.9
50	4.9	3 $\frac{7}{8}$	3 $\frac{7}{8}$	2 $\frac{1}{8}$	$\frac{7}{16}$	1.9	10.1	5.1
55	5.4	4 $\frac{1}{16}$	4 $\frac{1}{16}$	2 $\frac{1}{4}$	$\frac{15}{32}$	2.0	12.2	5.9
60	5.9	4 $\frac{1}{4}$	4 $\frac{1}{4}$	2 $\frac{3}{8}$	$\frac{31}{64}$	2.1	14.7	6.7
65	6.4	4 $\frac{7}{16}$	4 $\frac{7}{16}$	2 $\frac{13}{32}$	$\frac{1}{2}$	2.1	17.0	7.4
70	6.9	4 $\frac{5}{8}$	4 $\frac{5}{8}$	2 $\frac{7}{16}$	$\frac{33}{64}$	2.2	20.0	8.4
75	7.4	4 $\frac{13}{16}$	4 $\frac{13}{16}$	2 $\frac{15}{32}$	$\frac{17}{32}$	2.3	23.0	9.1
80	7.8	5	5	2 $\frac{1}{2}$	$\frac{35}{64}$	2.4	26.7	10.1
85	8.3	5 $\frac{3}{16}$	5 $\frac{3}{16}$	2 $\frac{9}{16}$	$\frac{9}{16}$	2.5	30.5	11.2
90	8.8	5 $\frac{3}{8}$	5 $\frac{3}{8}$	2 $\frac{5}{8}$	$\frac{9}{16}$	2.6	35.2	12.6
100	9.8	5 $\frac{3}{4}$	5 $\frac{3}{4}$	2 $\frac{3}{4}$	$\frac{9}{16}$	2.8	44.4	15.0

TABLE XIX
SIZES OF FULL AND COUNTERSUNK RIVET HEADS

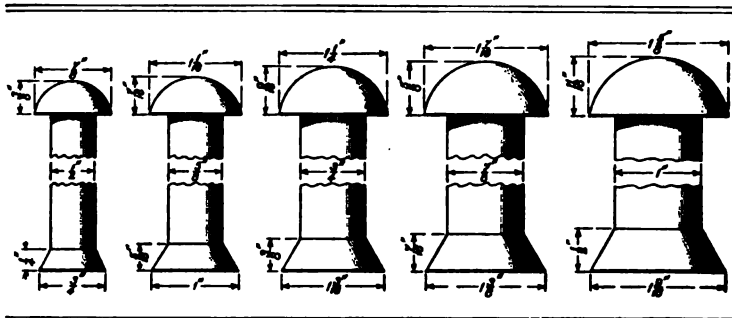


TABLE XX
ADDITIONAL LENGTH OF RIVET TO FORM ONE HEAD

Grip of Rivet Inches	Button Heads					Countersunk Rivets				
	Diameter of Rivet Inches					Diameter of Rivet Inches				
	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{7}{8}$	1	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{7}{8}$	1
$\frac{1}{2}$ to $1\frac{1}{4}$	1	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$\frac{1}{2}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$
$1\frac{3}{8}$ to 3	$1\frac{1}{8}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$\frac{5}{8}$	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{7}{8}$
$3\frac{1}{8}$ to $4\frac{5}{8}$	$1\frac{1}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$	$\frac{5}{8}$	1	1	1	1
$4\frac{3}{4}$ to 6	$1\frac{1}{4}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$	2	$\frac{3}{4}$	$1\frac{1}{8}$	$1\frac{1}{8}$	$1\frac{1}{8}$	$1\frac{1}{8}$

TABLE XXI
WEIGHT OF 100 RIVET HEADS

Diameter of Rivet, Inches	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{7}{8}$	1
Weight of 100 Heads Pounds	5.5	8.5	12.3	16.7	21.8

TABLE XXII
MULTIPLICATION TABLE FOR RIVET SPACING

Number of Spaces	Pitch of Rivets, in Inches						
	$1\frac{1}{2}$	$1\frac{3}{4}$	2	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{3}{4}$	3
	' "	' "	' "	' "	' "	' "	' "
2	0-3	0-3 $\frac{1}{4}$	0-3 $\frac{1}{2}$	0-3 $\frac{3}{4}$	0-4	0-4 $\frac{1}{4}$	0-4 $\frac{1}{2}$
3	0-4 $\frac{1}{2}$	0-4 $\frac{3}{4}$	0-5 $\frac{1}{4}$	0-5 $\frac{3}{4}$	0-6	0-6 $\frac{1}{4}$	0-6 $\frac{1}{2}$
4	0-6	0-6 $\frac{1}{2}$	0-7	0-7 $\frac{1}{2}$	0-8	0-8 $\frac{1}{4}$	0-9
5	0-7 $\frac{1}{2}$	0-8 $\frac{1}{4}$	0-8 $\frac{3}{4}$	0-9 $\frac{1}{4}$	0-10	0-10 $\frac{1}{4}$	0-11 $\frac{1}{4}$
6	0-9	0-9 $\frac{1}{4}$	0-10 $\frac{1}{2}$	0-11 $\frac{1}{4}$	1-0	1- $\frac{1}{4}$	1-1 $\frac{1}{4}$
7	0-10 $\frac{1}{2}$	0-11 $\frac{1}{4}$	1- $\frac{1}{4}$	1-1 $\frac{1}{2}$	1-2	1-2 $\frac{1}{4}$	1-3 $\frac{1}{4}$
8	1-0	1-1	1-2	1-3	1-4	1-5	1-6
9	1-1 $\frac{1}{2}$	1-2 $\frac{3}{4}$	1-3 $\frac{1}{2}$	1-4 $\frac{1}{4}$	1-6	1-7 $\frac{1}{4}$	1-8 $\frac{1}{4}$
10	1-3	1-4 $\frac{1}{4}$	1-5 $\frac{1}{2}$	1-6 $\frac{1}{4}$	1-8	1-9 $\frac{1}{4}$	1-10 $\frac{1}{4}$
11	1-4 $\frac{1}{2}$	1-5 $\frac{3}{4}$	1-7 $\frac{1}{4}$	1-8 $\frac{3}{4}$	1-10	1-11 $\frac{1}{4}$	2- $\frac{1}{4}$
12	1-6	1-7 $\frac{1}{2}$	1-9	1-10 $\frac{1}{2}$	2-0	2-1 $\frac{1}{4}$	2-3
13	1-7 $\frac{1}{2}$	1-9 $\frac{1}{4}$	1-10 $\frac{3}{4}$	2- $\frac{3}{4}$	2-2	2-3 $\frac{3}{4}$	2-5 $\frac{1}{4}$
14	1-9	1-10 $\frac{3}{4}$	2- $\frac{1}{2}$	2-2 $\frac{1}{4}$	2-4	2-5 $\frac{1}{4}$	2-7 $\frac{1}{4}$
15	1-10 $\frac{1}{2}$	2- $\frac{3}{4}$	2-2 $\frac{1}{2}$	2-4 $\frac{3}{4}$	2-6	2-7 $\frac{3}{4}$	2-9 $\frac{1}{4}$
16	2-0	2-2	2-4	2-6	2-8	2-10	3-0
17	2-1 $\frac{1}{2}$	2-3 $\frac{3}{4}$	2-5 $\frac{3}{4}$	2-7 $\frac{3}{4}$	2-10	3- $\frac{1}{4}$	3-2 $\frac{1}{4}$
18	2-3	2-5 $\frac{1}{4}$	2-7 $\frac{1}{2}$	2-9 $\frac{1}{4}$	3-0	3-2 $\frac{1}{4}$	3-4 $\frac{1}{4}$
19	2-4 $\frac{1}{2}$	2-6 $\frac{3}{4}$	2-9 $\frac{1}{4}$	2-11 $\frac{1}{4}$	3-2	3-4 $\frac{3}{4}$	3-6 $\frac{3}{4}$
20	2-6	2-8 $\frac{1}{2}$	2-11	3-1 $\frac{1}{2}$	3-4	3-6 $\frac{1}{2}$	3-9
21	2-7 $\frac{1}{2}$	2-10 $\frac{1}{4}$	3- $\frac{3}{4}$	3-3 $\frac{3}{4}$	3-6	3-8 $\frac{3}{4}$	3-11 $\frac{1}{4}$
22	2-9	2-11 $\frac{3}{4}$	3-2 $\frac{1}{2}$	3-5 $\frac{1}{4}$	3-8	3-10 $\frac{3}{4}$	4-1 $\frac{1}{4}$
23	2-10 $\frac{1}{2}$	3-1 $\frac{3}{4}$	3-4 $\frac{1}{2}$	3-7 $\frac{1}{4}$	3-10	4- $\frac{1}{4}$	4-3 $\frac{1}{4}$
24	3-0	3-3	3-6	3-9	4-0	4-3	4-6
25	3-1 $\frac{1}{2}$	3-4 $\frac{3}{4}$	3-7 $\frac{3}{4}$	3-10 $\frac{3}{4}$	4-2	4-5 $\frac{3}{4}$	4-8 $\frac{1}{4}$
26	3-3	3-6 $\frac{1}{4}$	3-9 $\frac{1}{4}$	4- $\frac{3}{4}$	4-4	4-7 $\frac{1}{4}$	4-10 $\frac{1}{4}$
27	3-4 $\frac{1}{2}$	3-7 $\frac{3}{4}$	3-11 $\frac{1}{4}$	4-2 $\frac{3}{4}$	4-6	4-9 $\frac{3}{4}$	5- $\frac{3}{4}$
28	3-6	3-9 $\frac{1}{2}$	4-1	4-4 $\frac{1}{2}$	4-8	4-11 $\frac{1}{2}$	5-3
29	3-7 $\frac{1}{2}$	3-11 $\frac{3}{4}$	4-2 $\frac{1}{2}$	4-6 $\frac{3}{4}$	4-10	5-1 $\frac{3}{4}$	5-5 $\frac{1}{4}$
30	3-9	4- $\frac{3}{4}$	4-4 $\frac{1}{2}$	4-8 $\frac{1}{4}$	5-0	5-3 $\frac{3}{4}$	5-7 $\frac{1}{4}$
31	3-10 $\frac{1}{2}$	4-2 $\frac{3}{4}$	4-6 $\frac{1}{2}$	4-10 $\frac{1}{4}$	5-2	5-5 $\frac{1}{4}$	5-9 $\frac{1}{4}$
32	4-0	4-4	4-8	5-0	5-4	5-8	6-0

BRIDGE TABLES

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TABLE XXII—(Continued)
MULTIPLICATION TABLE FOR RIVET SPACING

Number of Spaces	Pitch of Rivets, in Inches						
	$2\frac{1}{8}$	$2\frac{1}{2}$	$2\frac{3}{4}$	3	$3\frac{1}{2}$	4	$4\frac{1}{2}$
1	" "	" "	" "	" "	" "	" "	" "
2	0-4 $\frac{1}{8}$	0-5	0-5 $\frac{1}{2}$	0-5 $\frac{1}{2}$	0-5 $\frac{1}{2}$	0-6	0-6 $\frac{1}{2}$
3	0-7 $\frac{1}{8}$	0-7 $\frac{1}{2}$	0-7 $\frac{1}{2}$	0-8 $\frac{1}{2}$	0-8 $\frac{1}{2}$	0-9	0-9 $\frac{1}{2}$
4	0-9 $\frac{1}{2}$	0-10	0-10 $\frac{1}{2}$	0-11	0-11 $\frac{1}{2}$	1-0	1- $\frac{1}{2}$
5	0-11 $\frac{1}{4}$	1- $\frac{1}{2}$	1-1 $\frac{1}{2}$	1-1 $\frac{1}{2}$	1-2 $\frac{1}{8}$	1-3	1-3 $\frac{1}{8}$
6	1-2 $\frac{1}{4}$	1-3	1-3 $\frac{1}{2}$	1-4 $\frac{1}{2}$	1-5 $\frac{1}{2}$	1-6	1-6 $\frac{1}{2}$
7	1-4 $\frac{1}{8}$	1-5 $\frac{1}{2}$	1-6 $\frac{1}{2}$	1-7 $\frac{1}{2}$	1-8 $\frac{1}{2}$	1-9	1-9 $\frac{1}{2}$
8	1-7	1-8	1-9	1-10	1-11	2-0	2-1
9	1-9 $\frac{1}{8}$	1-10 $\frac{1}{2}$	1-11 $\frac{1}{2}$	2- $\frac{1}{4}$	2-1 $\frac{1}{8}$	2-3	2-4 $\frac{1}{8}$
10	1-11 $\frac{1}{4}$	2-1	2-2 $\frac{1}{2}$	2-3 $\frac{1}{2}$	2-4 $\frac{1}{2}$	2-6	2-7 $\frac{1}{2}$
11	2-2 $\frac{1}{2}$	2-3 $\frac{1}{2}$	2-4 $\frac{1}{2}$	2-6 $\frac{1}{2}$	2-7 $\frac{1}{2}$	2-9	2-10 $\frac{1}{2}$
12	2-4 $\frac{1}{2}$	2-6	2-7 $\frac{1}{2}$	2-9	2-10 $\frac{1}{2}$	3-0	3-1 $\frac{1}{2}$
13	2-6 $\frac{1}{8}$	2-8 $\frac{1}{2}$	2-10 $\frac{1}{2}$	2-11 $\frac{1}{2}$	3-1 $\frac{1}{8}$	3-3	3-4 $\frac{1}{8}$
14	2-9 $\frac{1}{4}$	2-11	3- $\frac{1}{4}$	3-2 $\frac{1}{2}$	3-4 $\frac{1}{2}$	3-6	3-7 $\frac{1}{2}$
15	2-11 $\frac{1}{8}$	3-1 $\frac{1}{2}$	3-3 $\frac{1}{2}$	3-5 $\frac{1}{2}$	3-7 $\frac{1}{2}$	3-9	3-10 $\frac{1}{2}$
16	3-2	3-4	3-6	3-8	3-10	4-0	4-2
17	3-4 $\frac{1}{8}$	3-6 $\frac{1}{2}$	3-8 $\frac{1}{2}$	3-10 $\frac{1}{2}$	4- $\frac{1}{4}$	4-3	4-5 $\frac{1}{8}$
18	3-6 $\frac{1}{2}$	3-9	3-11 $\frac{1}{2}$	4-1 $\frac{1}{2}$	4-3 $\frac{1}{2}$	4-6	4-8 $\frac{1}{2}$
19	3-9 $\frac{1}{4}$	3-11 $\frac{1}{2}$	4-1 $\frac{1}{2}$	4-4 $\frac{1}{2}$	4-6 $\frac{1}{2}$	4-9	4-11 $\frac{1}{2}$
20	3-11 $\frac{1}{2}$	4-2	4-4 $\frac{1}{2}$	4-7	4-9 $\frac{1}{2}$	5-0	5-2 $\frac{1}{2}$
21	4-1 $\frac{1}{4}$	4-4 $\frac{1}{2}$	4-7 $\frac{1}{2}$	4-9 $\frac{1}{2}$	5- $\frac{1}{4}$	5-3	5-5 $\frac{1}{4}$
22	4-4 $\frac{1}{2}$	4-7	4-9 $\frac{1}{2}$	5- $\frac{1}{2}$	5-3 $\frac{1}{2}$	5-6	5-8 $\frac{1}{2}$
23	4-6 $\frac{1}{2}$	4-9 $\frac{1}{2}$	5- $\frac{1}{2}$	5-3 $\frac{1}{2}$	5-6 $\frac{1}{2}$	5-9	5-11 $\frac{1}{2}$
24	4-9	5-0	5-3	5-6	5-9	6-0	6-3
25	4-11 $\frac{1}{4}$	5-2 $\frac{1}{2}$	5-5 $\frac{1}{2}$	5-8 $\frac{1}{2}$	5-11 $\frac{1}{4}$	6-3	6-6 $\frac{1}{4}$
26	5-1 $\frac{1}{2}$	5-5	5-8 $\frac{1}{2}$	5-11 $\frac{1}{2}$	6-2 $\frac{1}{2}$	6-6	6-9 $\frac{1}{2}$
27	5-4 $\frac{1}{2}$	5-7 $\frac{1}{2}$	5-10 $\frac{1}{2}$	6-2 $\frac{1}{2}$	6-5 $\frac{1}{2}$	6-9	7- $\frac{1}{2}$
28	5-6 $\frac{1}{2}$	5-10	6-1 $\frac{1}{2}$	6-5	6-8 $\frac{1}{2}$	7-0	7-3 $\frac{1}{2}$
29	5-8 $\frac{1}{4}$	6- $\frac{1}{2}$	6-4 $\frac{1}{2}$	6-7 $\frac{1}{2}$	6-11 $\frac{1}{4}$	7-3	7-6 $\frac{1}{4}$
30	5-11 $\frac{1}{4}$	6-3	6-6 $\frac{1}{2}$	6-10 $\frac{1}{2}$	7-2 $\frac{1}{2}$	7-6	7-9 $\frac{1}{2}$
31	6-1 $\frac{1}{2}$	6-5 $\frac{1}{2}$	6-9 $\frac{1}{2}$	7-1 $\frac{1}{2}$	7-5 $\frac{1}{2}$	7-9	8- $\frac{1}{2}$
32	6-4	6-8	7-0	7-4	7-8	8-0	8-4

BRIDGE TABLES

TABLE XXII—(Continued)









MULTIPLICATION TABLE FOR RIVET SPACING

Number of Spaces	Pitch of Rivets, in Inches						
	$3\frac{1}{4}$	$3\frac{3}{4}$	$3\frac{1}{2}$	$3\frac{5}{8}$	$3\frac{7}{8}$	4	$4\frac{1}{2}$
	" "	" "	" "	" "	" "	" "	" "
2	0-6 $\frac{1}{4}$	0-6 $\frac{3}{4}$	0-7	0-7 $\frac{1}{4}$	0-7 $\frac{3}{4}$	0-8	0-8 $\frac{1}{2}$
3	0-9 $\frac{1}{4}$	0-10 $\frac{1}{4}$	0-10 $\frac{1}{2}$	0-10 $\frac{3}{8}$	0-11 $\frac{1}{4}$	1-0	1- $\frac{1}{4}$
4	1-1	1-1 $\frac{1}{4}$	1-2	1-2 $\frac{1}{4}$	1-3	1-4	1-5
5	1-4 $\frac{1}{4}$	1-4 $\frac{3}{4}$	1-5 $\frac{1}{2}$	1-6 $\frac{1}{4}$	1-6 $\frac{3}{4}$	1-8	1-9 $\frac{1}{2}$
6	1-7 $\frac{1}{4}$	1-8 $\frac{1}{4}$	1-9	1-9 $\frac{1}{4}$	1-10 $\frac{1}{4}$	2-0	2-1 $\frac{1}{2}$
7	1-10 $\frac{1}{4}$	1-11 $\frac{1}{4}$	2- $\frac{1}{2}$	2-1 $\frac{3}{8}$	2-3 $\frac{1}{4}$	2-4	2-5 $\frac{1}{2}$
8	2-2	2-3	2-4	2-5	2-6	2-8	2-10
9	2-5 $\frac{1}{4}$	2-6 $\frac{1}{4}$	2-7 $\frac{1}{2}$	2-8 $\frac{3}{8}$	2-9 $\frac{1}{4}$	3-0	3-2 $\frac{1}{2}$
10	2-8 $\frac{1}{4}$	2-9 $\frac{1}{4}$	2-11	3- $\frac{1}{4}$	3-1 $\frac{1}{4}$	3-4	3-6 $\frac{1}{2}$
11	2-11 $\frac{1}{4}$	3-1 $\frac{1}{4}$	3-2 $\frac{1}{2}$	3-3 $\frac{1}{4}$	3-5 $\frac{1}{4}$	3-8	3-10 $\frac{1}{2}$
12	3-3	3-4 $\frac{1}{4}$	3-6	3-7 $\frac{1}{4}$	3-9	4-0	4-3
13	3-6 $\frac{1}{4}$	3-7 $\frac{1}{4}$	3-9 $\frac{1}{2}$	3-11 $\frac{1}{4}$	4- $\frac{1}{4}$	4-4	4-7 $\frac{1}{2}$
14	3-9 $\frac{1}{4}$	3-11 $\frac{1}{4}$	4-1	4-2 $\frac{1}{4}$	4-4 $\frac{1}{4}$	4-8	4-11 $\frac{1}{2}$
15	4- $\frac{3}{4}$	4-2 $\frac{3}{4}$	4-4 $\frac{1}{2}$	4-6 $\frac{3}{8}$	4-8 $\frac{1}{4}$	5-0	5-3 $\frac{1}{2}$
16	4-4	4-6	4-8	4-10	5-0	5-4	5-8
17	4-7 $\frac{1}{4}$	4-9 $\frac{1}{4}$	4-11 $\frac{1}{2}$	5-1 $\frac{3}{8}$	5-3 $\frac{1}{4}$	5-8	6- $\frac{1}{4}$
18	4-10 $\frac{1}{4}$	5- $\frac{3}{4}$	5-3	5-5 $\frac{1}{4}$	5-7 $\frac{1}{4}$	6-0	6-4 $\frac{1}{2}$
19	5-1 $\frac{3}{4}$	5-4 $\frac{3}{4}$	5-6 $\frac{1}{2}$	5-8 $\frac{3}{8}$	5-11 $\frac{1}{4}$	6-4	6-8 $\frac{3}{4}$
20	5-5	5-7 $\frac{1}{4}$	5-10	6- $\frac{1}{2}$	6-3	6-8	7-1
21	5-8 $\frac{1}{4}$	5-10 $\frac{3}{4}$	6-1 $\frac{1}{2}$	6-4 $\frac{3}{8}$	6-6 $\frac{1}{4}$	7-0	7-5 $\frac{1}{4}$
22	5-11 $\frac{1}{4}$	6-2 $\frac{1}{4}$	6-5	6-7 $\frac{1}{4}$	6-10 $\frac{1}{4}$	7-4	7-9 $\frac{1}{2}$
23	6-2 $\frac{3}{4}$	6-5 $\frac{3}{4}$	6-8 $\frac{1}{2}$	6-11 $\frac{3}{8}$	7-2 $\frac{1}{4}$	7-8	8-1 $\frac{1}{2}$
24	6-6	6-9	7-0	7-3	7-6	8-0	8-6
25	6-9 $\frac{1}{4}$	7- $\frac{3}{4}$	7-3 $\frac{1}{2}$	7-6 $\frac{3}{8}$	7-9 $\frac{1}{4}$	8-4	8-10 $\frac{1}{4}$
26	7- $\frac{1}{2}$	7-3 $\frac{3}{4}$	7-7	7-10 $\frac{1}{4}$	8-1 $\frac{1}{4}$	8-8	9-2 $\frac{1}{2}$
27	7-3 $\frac{3}{4}$	7-7 $\frac{3}{4}$	7-10 $\frac{1}{2}$	8-1 $\frac{3}{8}$	8-5 $\frac{1}{4}$	9-0	9-6 $\frac{3}{4}$
28	7-7	7-10 $\frac{3}{4}$	8-2	8-5 $\frac{3}{8}$	8-9	9-4	9-11
29	7-10 $\frac{1}{4}$	8-1 $\frac{3}{4}$	8-5 $\frac{1}{2}$	8-9 $\frac{3}{8}$	9- $\frac{3}{4}$	9-8	10-3 $\frac{1}{4}$
30	8-1 $\frac{1}{4}$	8-5 $\frac{1}{4}$	8-9	9- $\frac{1}{4}$	9-4 $\frac{1}{4}$	10-0	10-7 $\frac{1}{2}$
31	8-4 $\frac{1}{4}$	8-8 $\frac{3}{4}$	9- $\frac{1}{4}$	9-4 $\frac{3}{8}$	9-8 $\frac{1}{4}$	10-4	10-11 $\frac{1}{4}$
32	8-8	9-0	9-4	9-8	10-0	10-8	11-4












TABLE XXII—(Continued)
MULTIPLICATION TABLE FOR RIVET SPACING










Number of Spaces	Pitch of Rivets, in Inches						
	4½	4¾	5	5½	5¾	5¾	6
	' "	' "	' "	' "	' "	' "	' "
2	0-9	0-9½	0-10	0-10½	0-11	0-11½	1-0
3	1-1½	1-2½	1-3	1-3½	1-4½	1-5½	1-6
4	1-6	1-7	1-8	1-9	1-10	1-11	2-0
5	1-10½	1-11½	2-1	2-2½	2-3½	2-4½	2-6
6	2-3	2-4½	2-6	2-7½	2-9	2-10½	3-0
7	2-7½	2-9½	2-11	3-½	3-2½	3-4½	3-6
8	3-0	3-2	3-4	3-6	3-8	3-10	4-0
9	3-4½	3-6½	3-9	3-11½	4-1½	4-3½	4-6
10	3-9	3-11½	4-2	4-4½	4-7	4-9½	5-0
11	4-1½	4-4½	4-7	4-9½	5-½	5-3½	5-6
12	4-6	4-9	5-0	5-3	5-6	5-9	6-0
13	4-10½	5-1½	5-5	5-8½	5-11½	6-2½	6-6
14	5-3	5-6½	5-10	6-1½	6-5	6-8½	7-0
15	5-7½	5-11½	6-3	6-6½	6-10½	7-2½	7-6
16	6-0	6-4	6-8	7-0	7-4	7-8	8-0
17	6-4½	6-8½	7-1	7-5½	7-9½	8-1½	8-6
18	6-9	7-1½	7-6	7-10½	8-3	8-7½	9-0
19	7-1½	7-6½	7-11	8-3½	8-8½	9-1½	9-6
20	7-6	7-11	8-4	8-9	9-2	9-7	10-0
21	7-10½	8-3½	8-9	9-2½	9-7½	10-½	10-6
22	8-3	8-8½	9-2	9-7½	10-1	10-6½	11-0
23	8-7½	9-1½	9-7	10-½	10-6½	11-½	11-6
24	9-0	9-6	10-0	10-6	11-0	11-6	12-0
25	9-4½	9-10½	10-5	10-11½	11-5½	11-11½	12-6
26	9-9	10-3½	10-10	11-4½	11-11	12-5½	13-0
27	10-1½	10-8½	11-3	11-9½	12-4½	12-11½	13-6
28	10-6	11-1	11-8	12-3	12-10	13-5	14-0
29	10-10½	11-5½	12-1	12-8½	13-3½	13-10½	14-6
30	11-3	11-10½	12-6	13-1½	13-9	14-4½	15-0
31	11-7½	12-3½	12-11	13-6½	14-2½	14-10½	15-6
32	12-0	12-8	13-4	14-0	14-8	15-4	16-0

TABLE XXIII
CONVENTIONAL SIGNS FOR STRUCTURAL RIVETS

	SHOP	FIELD
Two full heads.....		
Countersunk inside and not chipped.....		
Countersunk outside and not chipped.....		
Countersunk both sides and not chipped.....		

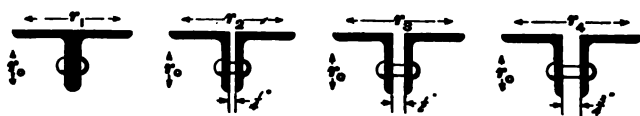
Where rivets must be chipped, the drawings should state the fact.

	INSIDE	OUTSIDE	BOTH SIDES
Flatten to $\frac{3}{8}$ inch high.			
(a)			
	SHOP	FIELD	
Two full heads.....			
Countersunk inside and chipped.....			
Countersunk outside and chipped.....			
Countersunk both sides and chipped.....			

	INSIDE	OUTSIDE	BOTH SIDES
Flatten to $\frac{1}{4}$ inch high or countersunk and not chipped.....			
Flatten to $\frac{1}{4}$ inch high.....			
Flatten to $\frac{3}{8}$ inch high.....			

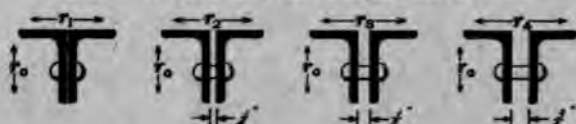
(b)

TABLE XXIV
RADI OF GYRATION FOR TWO ANGLES HAVING EQUAL
LEGS AND PLACED BACK TO BACK



Dimensions Inches	Thickness Inches	Area of Two Angles Square Inches	Radii of Gyration				
			r_0	r_1	r_2	r_3	r_4
2×2	$\frac{3}{16}$	1.42	.62	.84	.93	1.03	1.13
2×2	$\frac{7}{16}$	3.12	.59	.88	.98	1.08	1.19
2½×2½	$\frac{3}{16}$	1.80	.78	1.04	1.13	1.22	1.32
2½×2½	$\frac{1}{2}$	4.50	.74	1.10	1.19	1.29	1.40
3×3	$\frac{1}{4}$	2.88	.93	1.25	1.34	1.43	1.53
3×3	$\frac{3}{8}$	6.72	.88	1.32	1.41	1.51	1.62
3½×3½	$\frac{5}{16}$	4.18	1.08	1.47	1.56	1.65	1.74
3½×3½	$\frac{1}{2}$	10.06	1.02	1.55	1.64	1.74	1.85
4×4	$\frac{5}{16}$	4.80	1.24	1.67	1.76	1.85	1.94
4×4	$\frac{1}{2}$	11.68	1.18	1.75	1.84	1.94	2.04
6×6	$\frac{3}{8}$	8.72	1.88	2.49	2.58	2.67	2.76
6×6	1	22.00	1.80	2.59	2.68	2.77	2.87
8×8	$\frac{1}{2}$	15.50	2.50	3.32	3.40	3.49	3.58
8×8	1½	33.46	2.42	3.42	3.51	3.60	3.69

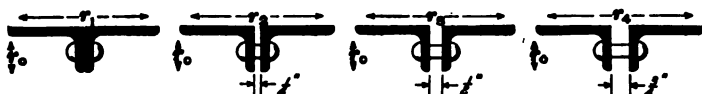
TABLE XXV

RADI OF GYRATION FOR TWO ANGLES HAVING UNEQUAL
LEGS AND PLACED BACK TO BACK

Dimensions Inches	Thickness Inch	Area of Two Angles Square Inches	Radi of Gyration				
			r_0	r_1	r_2	r_3	r_4
$2\frac{1}{2} \times 2$	$\frac{3}{16}$	1.62	.79	.79	.88	.97	1.07
$2\frac{1}{2} \times 2$	$\frac{1}{2}$	4.00	.75	.84	.94	1.04	1.15
$3 \times 2\frac{1}{2}$	$\frac{1}{2}$	2.62	.95	1.00	1.09	1.18	1.28
$3 \times 2\frac{1}{2}$	$\frac{3}{16}$	5.56	.91	1.05	1.15	1.25	1.35
$3\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{2}$	2.88	1.12	.96	1.04	1.13	1.23
$3\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{16}$	7.10	1.06	1.03	1.13	1.23	1.33
$3\frac{1}{2} \times 3$	$\frac{3}{16}$	3.86	1.10	1.22	1.31	1.40	1.49
$3\frac{1}{2} \times 3$	$\frac{1}{16}$	9.24	1.04	1.30	1.40	1.50	1.60
4×3	$\frac{3}{16}$	4.18	1.27	1.17	1.26	1.35	1.44
4×3	$\frac{1}{16}$	10.06	1.21	1.25	1.35	1.45	1.55
5×3	$\frac{3}{16}$	4.80	1.61	1.09	1.17	1.26	1.35
5×3	$\frac{1}{16}$	11.68	1.55	1.18	1.27	1.37	1.47
$5 \times 3\frac{1}{2}$	$\frac{3}{16}$	5.12	1.61	1.33	1.41	1.50	1.59
$5 \times 3\frac{1}{2}$	$\frac{1}{2}$	13.34	1.53	1.42	1.51	1.61	1.71
$6 \times 3\frac{1}{2}$	$\frac{1}{2}$	6.84	1.94	1.26	1.34	1.43	1.53
$6 \times 3\frac{1}{2}$	1	17.00	1.85	1.37	1.46	1.56	1.67
6×4	$\frac{1}{2}$	7.22	1.93	1.50	1.58	1.67	1.76
6×4	1	18.00	1.85	1.60	1.69	1.79	1.89
$7 \times 3\frac{1}{2}$	$\frac{7}{16}$	8.80	2.26	1.21	1.29	1.39	1.47
$7 \times 3\frac{1}{2}$	1	19.00	2.19	1.31	1.40	1.50	1.60

TABLE XXVI

RADII OF GYRATION FOR TWO ANGLES HAVING UNEQUAL LEGS AND PLACED BACK TO BACK

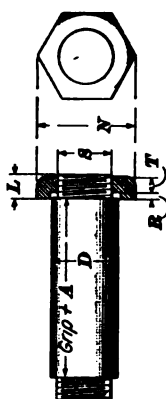


Dimensions Inches	Thickness Inch	Area of Two Angles Square Inches	Radii of Gyration				
			r_0	r_1	r_2	r_3	r_4
$2\frac{1}{2} \times 2$	$\frac{3}{16}$	1.62	.60	1.10	1.19	1.28	1.39
$2\frac{1}{2} \times 2$	$\frac{1}{2}$	4.00	.56	1.16	1.25	1.35	1.46
$3 \times 2\frac{1}{2}$	$\frac{1}{4}$	2.62	.75	1.31	1.40	1.50	1.59
$3 \times 2\frac{1}{2}$	$\frac{3}{16}$	5.56	.72	1.37	1.46	1.56	1.66
$3\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{4}$	2.88	.74	1.58	1.67	1.76	1.86
$3\frac{1}{2} \times 2\frac{1}{2}$	$\frac{3}{16}$	7.10	.67	1.66	1.76	1.86	1.96
$3\frac{1}{2} \times 3$	$\frac{5}{16}$	3.86	.90	1.52	1.61	1.71	1.80
$3\frac{1}{2} \times 3$	$\frac{3}{16}$	9.24	.85	1.61	1.71	1.81	1.91
4×3	$\frac{5}{16}$	4.18	.89	1.79	1.88	1.97	2.07
4×3	$\frac{3}{16}$	10.06	.83	1.88	1.98	2.08	2.18
5×3	$\frac{5}{16}$	4.80	.85	2.33	2.42	2.51	2.61
5×3	$\frac{3}{16}$	11.68	.80	2.42	2.52	2.62	2.72
$5 \times 3\frac{1}{2}$	$\frac{5}{16}$	5.12	1.03	2.26	2.35	2.44	2.54
$5 \times 3\frac{1}{2}$	$\frac{3}{16}$	13.34	.96	2.36	2.45	2.55	2.65
$6 \times 3\frac{1}{2}$	$\frac{3}{8}$	6.84	.99	2.81	2.90	3.00	3.10
$6 \times 3\frac{1}{2}$	1	17.00	.92	2.93	3.03	3.13	3.23
6×4	$\frac{3}{8}$	7.22	1.17	2.74	2.83	2.92	3.01
6×4	1	18.00	1.09	2.85	2.94	3.04	3.14
$7 \times 3\frac{1}{2}$	$\frac{7}{16}$	8.80	.95	3.37	3.46	3.56	3.66
$7 \times 3\frac{1}{2}$	1	19.00	.89	3.48	3.58	3.68	3.78

TABLE XXVII
AREAS TO BE DEDUCTED FOR RIVET HOLES

Thickness of Plate Inches	Diameter of Rivets, in Inches				
	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{7}{8}$	1
$\frac{1}{4}$.156	.188	.219	.250	.281
$\frac{5}{16}$.195	.234	.273	.313	.352
$\frac{3}{8}$.234	.281	.328	.375	.422
$\frac{7}{16}$.273	.328	.383	.438	.492
$\frac{1}{2}$.313	.375	.438	.500	.563
$\frac{9}{16}$.352	.422	.492	.563	.633
$\frac{5}{8}$.391	.469	.547	.625	.703
$\frac{11}{16}$.430	.516	.602	.688	.773
$\frac{3}{4}$.469	.563	.656	.750	.844
$\frac{13}{16}$.508	.609	.711	.813	.914
$\frac{7}{8}$.547	.656	.766	.875	.984
$\frac{15}{16}$.586	.703	.820	.938	1.055
1	.625	.750	.875	1.000	1.125
$1\frac{1}{8}$.703	.844	.984	1.125	1.266
$1\frac{1}{4}$.781	.938	1.094	1.250	1.406
$1\frac{3}{8}$.859	1.031	1.203	1.375	1.547
$1\frac{1}{2}$.937	1.125	1.313	1.500	1.688
Counter- sunk rivets	.031	.059	.082	.109	.141

TABLE XXVIII
DIMENSIONS OF SCREW ENDS AND NUTS FOR PINS, IN INCHES



BRIDGE TABLES

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Diameter of Pin D	Diameter of Screw S	Length of Screw L	Thickness of Nut T	Recess in Nut R	Long Diameter of Nut N	Weight of Nut Pounds	Add to Grip A
2 to $2\frac{1}{8}$	$1\frac{1}{2}$	$1\frac{1}{2}$	1	$\frac{3}{8}$	$3\frac{1}{2}$	2.5	$\frac{1}{4}$
$2\frac{1}{8}$ to $2\frac{1}{4}$	2	$1\frac{1}{2}$	1	$\frac{3}{8}$	$4\frac{1}{8}$	2.5	$\frac{1}{4}$
$2\frac{1}{4}$ to $3\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{2}$	1	$\frac{3}{8}$	$5\frac{1}{8}$	3.0	$\frac{1}{4}$
$3\frac{1}{8}$ to 4	3	$1\frac{7}{8}$	$1\frac{1}{4}$	$\frac{1}{2}$	$5\frac{1}{2}$	5.5	$\frac{1}{2}$
$4\frac{1}{8}$ to $4\frac{1}{4}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$1\frac{1}{4}$	$\frac{1}{2}$	$6\frac{1}{8}$	7.0	$\frac{1}{2}$
$4\frac{1}{4}$ to $5\frac{1}{4}$	4	$1\frac{7}{8}$	$1\frac{1}{4}$	$\frac{1}{2}$	$7\frac{1}{2}$	8.5	$\frac{1}{2}$
$5\frac{3}{8}$ to 6	$4\frac{1}{2}$	$1\frac{7}{8}$	$1\frac{1}{4}$	$\frac{1}{2}$	$8\frac{1}{2}$	11.0	$\frac{1}{2}$
$6\frac{1}{8}$ to $6\frac{1}{4}$	5	$2\frac{3}{8}$	$1\frac{1}{2}$	$\frac{3}{4}$	9	12.0	$\frac{3}{4}$
$6\frac{1}{4}$ to $7\frac{1}{4}$	$5\frac{1}{2}$	$2\frac{3}{8}$	$1\frac{1}{2}$	$\frac{3}{4}$	$9\frac{1}{2}$	13.5	$\frac{3}{4}$
$7\frac{1}{8}$ to 8	6	$2\frac{3}{8}$	$1\frac{1}{2}$	$\frac{3}{4}$	$10\frac{3}{8}$	17.0	$\frac{3}{4}$

TABLE XXIX
DIMENSIONS OF LATERAL PINS, IN INCHES



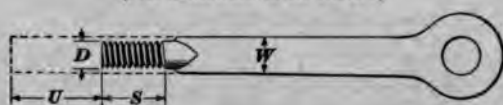
Diameter of Pin D	Diameter of Head H	Thickness of Head T	Diameter of Cotter Pin d	Length of Cotter Pin C	Length of End E
1	1 $\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	1 $\frac{3}{4}$	$\frac{3}{8}$
1 $\frac{1}{8}$	1 $\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{4}$	2	$\frac{3}{8}$
1 $\frac{1}{2}$	1 $\frac{3}{4}$	$\frac{1}{4}$	$\frac{5}{16}$	2 $\frac{1}{2}$	$\frac{1}{2}$
1 $\frac{3}{8}$	2	$\frac{1}{4}$	$\frac{5}{16}$	2 $\frac{3}{4}$	$\frac{1}{2}$
2	2 $\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	3	1
2 $\frac{1}{8}$	2 $\frac{3}{4}$	$\frac{3}{8}$	$\frac{3}{8}$	3 $\frac{1}{4}$	1
2 $\frac{3}{8}$	2 $\frac{7}{8}$	$\frac{3}{8}$	$\frac{7}{16}$	3 $\frac{3}{4}$	1 $\frac{1}{8}$
2 $\frac{7}{8}$	3 $\frac{1}{4}$	$\frac{3}{8}$	$\frac{7}{16}$	4	1 $\frac{1}{8}$
3	3 $\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	5	1 $\frac{3}{8}$

TABLE XXX
STEEL EYEBARS



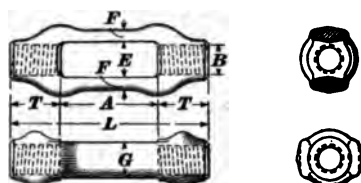
Width of Body of Bar <i>W</i> Inches	Minimum Thickness of Bar <i>t</i> Inch	Diameter of Head of Bar <i>H</i> Inches	Diameter of Largest Pinhole <i>P</i> Inches	Additional Length to Form Head <i>U</i>
				' "
2	$\frac{1}{2}$	$4\frac{1}{2}$	$1\frac{7}{8}$	0- $7\frac{1}{2}$
2	$\frac{1}{2}$	$5\frac{1}{2}$	$2\frac{7}{8}$	1- $\frac{1}{2}$
$2\frac{1}{2}$	$\frac{1}{2}$	$5\frac{1}{2}$	$2\frac{7}{8}$	0- $9\frac{1}{2}$
$2\frac{1}{2}$	$\frac{1}{2}$	$6\frac{1}{2}$	$3\frac{7}{8}$	1- $1\frac{1}{2}$
3	$\frac{3}{4}$	$6\frac{1}{2}$	$2\frac{1}{2}$	0- $10\frac{1}{2}$
3	$\frac{3}{4}$	8	4	1- $5\frac{1}{2}$
3	$\frac{3}{4}$	9	5	1- $10\frac{1}{2}$
4	$\frac{3}{4}$	$9\frac{1}{2}$	$4\frac{7}{8}$	1- $5\frac{1}{2}$
4	$\frac{3}{4}$	$10\frac{1}{2}$	$5\frac{7}{8}$	1- 9
4	$\frac{3}{4}$	$11\frac{1}{2}$	$6\frac{7}{8}$	2- $3\frac{1}{2}$
5	$\frac{3}{4}$	$11\frac{1}{2}$	$4\frac{7}{8}$	1- 8
5	$\frac{3}{4}$	$12\frac{1}{2}$	$5\frac{7}{8}$	2- 0
5	1	13	$6\frac{7}{8}$	2- $3\frac{1}{2}$
5	1	14	$7\frac{7}{8}$	2- 8
6	$\frac{7}{8}$	$13\frac{1}{2}$	$5\frac{1}{2}$	1- $9\frac{1}{2}$
6	$\frac{7}{8}$	$14\frac{1}{2}$	$6\frac{1}{2}$	2- 3
6	1	$15\frac{1}{2}$	$7\frac{1}{2}$	2- $7\frac{1}{2}$
7	$1\frac{1}{8}$	$15\frac{1}{2}$	$5\frac{7}{8}$	2- 2
7	$1\frac{1}{8}$	17	$7\frac{7}{8}$	2- 8
8	1	17	$5\frac{3}{4}$	2- $1\frac{1}{2}$
8	1	18	$6\frac{3}{4}$	2- $6\frac{1}{2}$
8	1	19	8	2- 11

TABLE XXXI
ADJUSTABLE EYEBARS
(All dimensions in inches)



Width of Bar W	Thickness of Bar t	Diameter of Upset End D	Length of Screw S	Additional Length Required to Form Screw End U
2	1	2	$5\frac{1}{2}$	6
3	$\frac{7}{8}$	$2\frac{1}{4}$	$6\frac{1}{2}$	$11\frac{1}{2}$
3	1	$2\frac{1}{2}$	$6\frac{1}{2}$	$11\frac{1}{4}$
3	$1\frac{1}{4}$	$2\frac{3}{4}$	7	11
3	$1\frac{1}{2}$	3	7	10
4	$\frac{3}{4}$	$2\frac{1}{2}$	$6\frac{1}{2}$	$12\frac{3}{4}$
4	1	$2\frac{3}{4}$	7	11
4	$1\frac{1}{4}$	$3\frac{1}{4}$	$7\frac{1}{4}$	11
4	$1\frac{1}{2}$	$3\frac{1}{2}$	$7\frac{1}{2}$	10
4	$1\frac{3}{4}$	$3\frac{3}{4}$	$7\frac{1}{2}$	$9\frac{1}{2}$
5	$\frac{3}{4}$	$2\frac{3}{4}$	7	11
5	1	$3\frac{1}{4}$	$7\frac{1}{4}$	$10\frac{1}{2}$
5	$1\frac{1}{4}$	$3\frac{1}{2}$	$7\frac{1}{2}$	$9\frac{1}{2}$
5	$1\frac{1}{2}$	$3\frac{3}{4}$	$7\frac{1}{2}$	$9\frac{3}{4}$
5	$1\frac{3}{4}$	4	$7\frac{1}{2}$	10
6	$1\frac{1}{8}$	$3\frac{3}{4}$	$7\frac{1}{2}$	10
6	$1\frac{1}{4}$	$3\frac{1}{2}$	$7\frac{1}{2}$	9
6	$1\frac{1}{2}$	4	$7\frac{1}{2}$	9

TABLE XXXII
PRESSED WROUGHT-IRON TURNBUCKLES



Diameter of Screw Inches <i>B</i>	Dimensions, in Inches					Weight of One Turn- buckle Pounds
	<i>L</i>	<i>T</i>	<i>E</i>	<i>F</i>	<i>G</i>	
$\frac{7}{8}$	$8\frac{5}{8}$	$1\frac{5}{16}$	$1\frac{1}{4}$	$\frac{3}{8}$	$1\frac{1}{4}$	$2\frac{1}{2}$
1	9	$1\frac{1}{2}$	$1\frac{5}{16}$	$\frac{7}{16}$	$1\frac{1}{4}$	$3\frac{1}{2}$
$1\frac{1}{8}$	$9\frac{3}{8}$	$1\frac{11}{16}$	$1\frac{7}{16}$	$\frac{1}{2}$	$1\frac{1}{4}$	4
$1\frac{1}{4}$	$9\frac{3}{4}$	$1\frac{7}{8}$	$1\frac{9}{16}$	$\frac{1}{2}$	$1\frac{1}{2}$	$5\frac{1}{4}$
$1\frac{3}{8}$	$10\frac{1}{8}$	$2\frac{1}{16}$	$1\frac{11}{16}$	$\frac{1}{2}$	$1\frac{5}{8}$	6
$1\frac{1}{2}$	$10\frac{1}{2}$	$2\frac{1}{4}$	$1\frac{3}{4}$	$\frac{5}{8}$	$1\frac{3}{4}$	7
$1\frac{5}{8}$	$10\frac{7}{8}$	$2\frac{7}{16}$	2	$\frac{5}{8}$	$1\frac{3}{4}$	$8\frac{1}{2}$
$1\frac{3}{4}$	$11\frac{1}{4}$	$2\frac{5}{8}$	$2\frac{1}{8}$	$\frac{5}{8}$	2	10
$1\frac{7}{8}$	$11\frac{5}{8}$	$2\frac{13}{16}$	$2\frac{3}{16}$	$\frac{11}{16}$	2	$11\frac{1}{2}$
2	12	3	$2\frac{3}{8}$	$\frac{11}{16}$	$2\frac{1}{4}$	13
$2\frac{1}{8}$	$12\frac{3}{8}$	$3\frac{3}{16}$	$2\frac{1}{2}$	$\frac{23}{32}$	$2\frac{1}{2}$	15
$2\frac{1}{4}$	$12\frac{1}{2}$	$3\frac{3}{8}$	$2\frac{11}{16}$	$\frac{13}{16}$	$2\frac{1}{2}$	18
$2\frac{3}{8}$	$13\frac{1}{8}$	$3\frac{9}{16}$	$2\frac{3}{4}$	$\frac{13}{16}$	$2\frac{3}{4}$	20
$2\frac{1}{2}$	$13\frac{1}{2}$	$3\frac{3}{4}$	$3\frac{1}{16}$	$\frac{27}{32}$	3	24
$2\frac{5}{8}$	$13\frac{7}{8}$	$3\frac{15}{16}$	$3\frac{1}{8}$	$\frac{15}{16}$	3	28
$2\frac{3}{4}$	$14\frac{1}{4}$	$4\frac{1}{8}$	$3\frac{1}{4}$	$\frac{15}{16}$	$3\frac{1}{4}$	30
$2\frac{7}{8}$	$14\frac{5}{8}$	$4\frac{5}{16}$	$3\frac{7}{16}$	$1\frac{1}{32}$	$3\frac{1}{4}$	34
3	15	$4\frac{1}{2}$	$3\frac{5}{8}$	$1\frac{1}{32}$	$3\frac{1}{2}$	38
$3\frac{1}{4}$	$15\frac{3}{4}$	$4\frac{7}{8}$	$3\frac{7}{8}$	$1\frac{1}{16}$	4	50
$3\frac{1}{2}$	$16\frac{1}{2}$	$5\frac{1}{4}$	$4\frac{1}{4}$	$1\frac{7}{32}$	4	65
$3\frac{3}{4}$	18	6	$4\frac{7}{16}$	$1\frac{5}{16}$	5	82
4	18	6	$4\frac{5}{8}$	$1\frac{7}{16}$	5	100

TABLE XXXV
WORKING STRESSES FOR COMPRESSION

$\frac{l}{r}$	$\frac{16,000}{1 + \frac{P^2}{18,000 r^2}}$	$\frac{l}{r}$	$\frac{16,000}{1 + \frac{P^2}{18,000 r^2}}$	$\frac{l}{r}$	$\frac{16,000}{1 + \frac{P^2}{18,000 r^2}}$
16	15,780	51	13,980	86	11,340
17	15,750	52	13,910	87	11,260
18	15,720	53	13,840	88	11,190
19	15,690	54	13,770	89	11,110
20	15,650	55	13,700	90	11,030
21	15,620	56	13,630	91	10,960
22	15,580	57	13,550	92	10,880
23	15,530	58	13,480	93	10,810
24	15,500	59	13,410	94	10,730
25	15,460	60	13,330	95	10,660
26	15,420	61	13,260	96	10,580
27	15,380	62	13,180	97	10,510
28	15,330	63	13,110	98	10,430
29	15,290	64	13,030	99	10,360
30	15,240	65	12,960	100	10,290
31	15,190	66	12,880	101	10,210
32	15,140	67	12,810	102	10,140
33	15,090	68	12,730	103	10,070
34	15,030	69	12,650	104	9,990
35	14,980	70	12,580	105	9,920
36	14,930	71	12,500	106	9,850
37	14,870	72	12,420	107	9,780
38	14,810	73	12,340	108	9,710
39	14,750	74	12,270	109	9,640
40	14,690	75	12,190	110	9,570
41	14,630	76	12,110	111	9,500
42	14,570	77	12,040	112	9,430
43	14,510	78	11,960	113	9,360
44	14,450	79	11,880	114	9,290
45	14,380	80	11,800	115	9,220
46	14,320	81	11,730	116	9,160
47	14,250	82	11,650	117	9,090
48	14,180	83	11,570	118	9,020
49	14,120	84	11,490	119	8,950
50	14,050	85	11,420	120	8,890

TABLE XXXVI
WORKING STRESSES FOR SHEAR

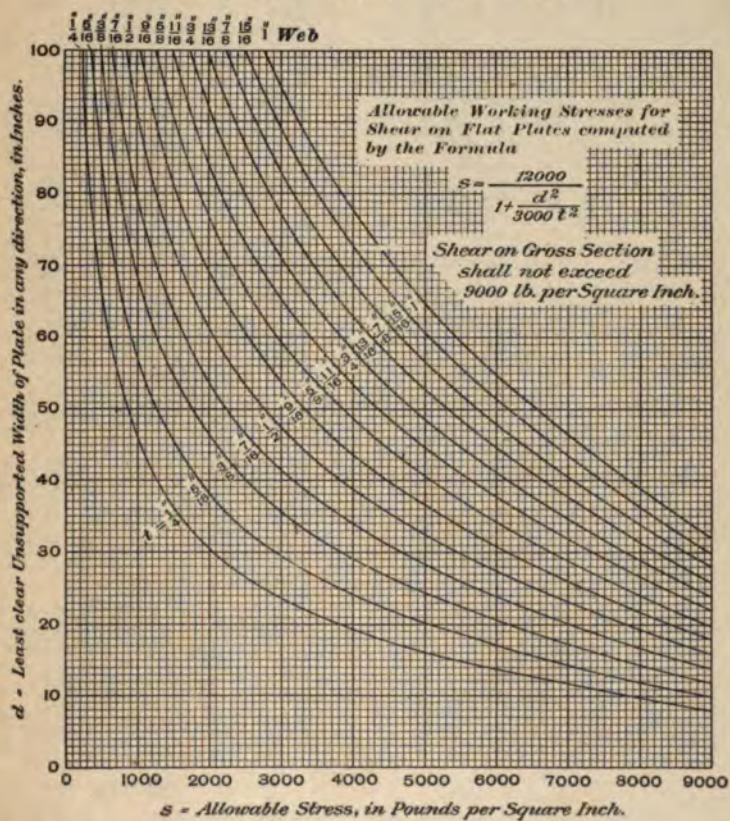


TABLE XXXVII
SHEARING AND BEARING VALUES OF RIVETS, IN POUNDS

Diam- eter of Rivet Inch	Area of Rivet Square Inch	Shear Values at 6,000 Pounds per Square Inch		Bearing Values for Different Thicknesses of Plate, in Inches, at 12,000 Pounds per Square Inch											
		Single Shear	Double Shear	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
				$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
$\frac{1}{8}$.1963	1,180	2,360	1,500	1,880	2,250	2,630	3,000							
$\frac{3}{16}$.3068	1,840	3,680	1,880	2,340	2,810	3,280	3,750	4,220	4,690					
$\frac{1}{4}$.4418	2,650	5,300	2,250	2,810	3,380	3,940	4,500	5,060	5,630	6,190	6,750			
$\frac{5}{16}$.6013	3,610	7,220	2,630	3,280	3,940	4,590	5,250	5,910	6,560	7,220	7,880	8,530	9,190	
1	.7854	4,710	9,420	3,000	3,750	4,500	5,250	6,000	6,750	7,500	8,250	9,000	9,750	10,500	11,250

TABLE XXXVIII
SHEARING AND BEARING VALUES OF RIVETS, IN POUNDS

Diam- eter of Rivet Inch	Area of Rivet Square Inch	Shear Values at 7,500 Pounds per Square Inch		Bearing Values for Different Thicknesses of Plate, in Inches, at 15,000 Pounds per Square Inch											
		Single Shear	Double Shear	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
				$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
$\frac{1}{8}$.1963	1,470	2,940	1,880	2,340	2,810	3,280	3,750							
$\frac{3}{16}$.3068	2,300	4,600	2,340	2,930	3,520	4,100	4,690	5,280	5,860					
$\frac{1}{4}$.4418	3,310	6,630	2,810	3,520	4,220	4,920	5,630	6,330	7,030	7,720	8,440			
$\frac{5}{16}$.6013	4,510	9,020	3,280	4,100	4,920	5,740	6,560	7,380	8,200	9,030	9,850	10,670	11,480	
1	.7854	5,890	11,780	3,750	4,690	5,630	6,560	7,500	8,440	9,380	10,310	11,250	12,190	13,130	14,060

TABLE XXXIX
SHEARING AND BEARING VALUES OF RIVETS, IN POUNDS

Diam- eter of Rivet Inch	Area of Rivet Square Inch	Shear Values at 9,000 Pounds per Square Inch		Bearing Values for Different Thicknesses of Plate, in Inches, at 18,000 Pounds per Square Inch											
		Single Shear	Double Shear	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
				$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
$\frac{1}{8}$.1963	1,770	3,530	2,250	2,810	3,380	3,940	4,500							
$\frac{1}{4}$.3068	2,760	5,520	2,810	3,520	4,220	4,920	5,630	6,330	7,030					
$\frac{3}{8}$.4418	3,980	7,950	3,380	4,220	5,060	5,910	6,750	7,590	8,440	9,280	10,130			
$\frac{1}{2}$.6013	5,410	10,820	3,940	4,920	5,910	6,890	7,880	8,860	9,840	10,830	11,810	12,800	13,780	
1	.7854	7,070	14,140	4,500	5,630	6,750	7,880	9,000	10,130	11,250	12,380	13,500	14,630	15,750	16,880

TABLE XL
SHEARING AND BEARING VALUES OF RIVETS, IN POUNDS

Diam- eter of Rivet Inch	Area of Rivet Square Inch	Shear Values at 11,000 Pounds per Square Inch		Bearing Values for Different Thicknesses of Plate, in Inches, at 22,000 Pounds per Square Inch											
		Single Shear	Double Shear	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
				$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
$\frac{1}{8}$.1963	2,160	4,320	2,750	3,440	4,130	4,820	5,500							
$\frac{1}{4}$.3068	3,370	6,750	3,440	4,300	5,160	6,020	6,880	7,740	8,600					
$\frac{3}{8}$.4418	4,860	9,720	4,130	5,160	6,190	7,220	8,250	9,280	10,320	11,340	12,380			
$\frac{1}{2}$.6013	6,610	13,230	4,810	6,020	7,220	8,430	9,630	10,840	12,040	13,240	14,440	15,640	16,840	
1	.7854	8,640	17,280	5,500	6,880	8,250	9,630	11,000	12,380	13,750	15,130	16,500	17,880	19,250	20,630

TABLE XLI
BEARING VALUES AND RESISTING MOMENTS OF PINS

Diameter of Pin Inches	Area of Pin Square Inches	Bearing Values, in Pounds, for 1 Inch Thickness of Plate			Moments, in Inch-Pounds, for Extreme Fiber Stresses of					
		15,000 Pounds per Square Inch	18,000 Pounds per Square Inch	22,000 Pounds per Square Inch	15,000 Pounds per Square Inch	18,000 Pounds per Square Inch	20,000 Pounds per Square Inch	22,000 Pounds per Square Inch	25,000 Pounds per Square Inch	
		3	4	5	6	7	8	9	10	
1										
1	.785	15,000	18,000	22,000	1,470	1,770	1,965	2,160	2,450	
1½	1.227	18,750	22,500	27,500	2,880	3,450	3,830	4,220	4,790	
1½	1.767	22,500	27,000	33,000	4,970	5,960	6,630	7,290	8,286	
1½	2.405	26,250	31,500	38,500	7,890	9,470	10,500	11,570	13,200	
2	3.142	30,000	36,000	44,000	11,800	14,100	15,700	17,280	19,600	
2½	3.547	31,880	38,250	46,750	14,100	17,000	18,800	20,730	23,600	
2½	3.976	33,750	40,500	49,500	16,800	20,100	22,400	24,600	28,000	
2½	4.430	35,630	42,750	52,250	19,700	23,700	26,300	28,900	32,900	
2½	4.909	37,500	45,000	55,000	23,000	27,600	30,700	33,700	38,400	
2½	5.412	39,380	47,250	57,750	26,600	32,000	35,500	39,000	44,400	
2½	5.940	41,250	49,500	60,500	30,600	36,800	40,800	44,900	51,000	
2½	6.492	43,130	51,750	63,250	35,000	42,000	46,700	51,300	58,300	
3	7.069	45,000	54,000	66,000	39,800	47,700	53,000	58,300	66,300	
3½	7.670	46,880	56,250	68,750	44,900	53,900	59,900	65,900	74,900	
3½	8.296	48,750	58,500	71,500	50,600	60,700	67,400	74,100	84,300	
3½	8.946	50,630	60,750	74,250	56,600	67,900	75,500	83,000	94,400	

3½	9.621	52,500	63,000	77,000	63,100	75,800	84,200	92,600	105,200
3½	10.32	54,380	65,250	79,750	70,100	84,200	93,500	102,900	116,900
3¾	11.05	56,250	67,500	82,500	77,700	93,200	103,500	113,900	129,400
3¾	11.79	58,130	69,750	85,250	85,700	102,800	114,200	125,600	142,800
4	12.57	60,000	72,000	88,000	94,200	113,100	125,700	138,200	157,100
4½	13.36	61,880	74,250	90,750	103,400	124,000	137,800	151,600	172,300
4½	14.19	63,750	76,500	93,500	113,000	135,700	150,700	165,800	188,400
4¾	15.03	65,630	78,750	96,250	113,300	148,000	164,400	180,800	205,500
4¾	15.90	67,500	81,000	99,000	134,200	161,000	178,900	196,800	223,700
4¾	16.80	69,380	83,250	101,750	145,700	174,800	194,300	213,700	242,800
4¾	17.72	71,250	85,500	104,500	157,800	189,400	210,400	231,500	263,000
4¾	18.67	73,130	87,750	107,250	170,000	204,700	227,500	250,200	284,400
5	19.64	75,000	90,000	110,000	184,100	220,900	245,400	270,000	306,800
5½	21.05	78,750	94,500	115,500	213,100	255,700	284,100	312,500	355,200
5½	23.76	82,500	99,000	121,000	245,000	294,000	326,700	359,300	408,300
5½	25.97	86,250	103,500	126,500	280,000	335,900	373,300	410,600	466,600
6	28.27	90,000	108,000	132,000	318,100	381,700	424,100	466,500	530,200
6½	30.68	93,750	112,500	137,500	359,500	431,400	479,400	527,300	599,200
6½	33.18	97,500	117,000	143,000	404,400	485,400	539,200	593,100	674,000
6¾	35.79	101,250	121,500	148,500	452,900	543,500	603,900	664,200	754,800
7	38.48	105,000	126,000	154,000	505,100	606,100	673,500	740,800	841,900
7½	41.28	108,750	130,500	159,500	561,200	673,400	748,200	823,000	935,300
7½	44.18	112,500	135,000	165,000	621,300	745,500	828,400	911,200	1,035,400
7¾	47.17	116,250	139,500	170,500	685,500	822,600	914,000	1,005,300	1,142,500
8	50.27	120,000	144,000	176,000	754,000	904,800	1,005,300	1,105,800	1,256,600

TABLE XLI
BEARING VALUES AND RESISTING MOMENTS OF PINS

Diameter of Pin Inches	Area of Pin Square Inches	Bearing Values, in Pounds, for 1 Inch Thickness of Plate				Moments, in Inch-Pounds, for Extreme Fiber Stresses of				
		15,000 Pounds per Square Inch	18,000 Pounds per Square Inch	22,000 Pounds per Square Inch	5	15,000 Pounds per Square Inch	18,000 Pounds per Square Inch	20,000 Pounds per Square Inch	22,000 Pounds per Square Inch	25,000 Pounds per Square Inch
1	2	3	4	5	6	7	8	9	10	
1	.785	15,000	18,000	22,000	1,470	1,770	1,960	2,160	2,450	
1 1/4	1.227	18,750	22,500	27,500	2,880	3,450	3,830	4,220	4,790	
1 1/2	1.767	22,500	27,000	33,000	4,970	5,960	6,630	7,290	8,280	
1 3/4	2.405	26,250	31,500	38,500	7,890	9,470	10,500	11,570	13,200	
2	3.142	30,000	36,000	44,000	11,800	14,100	15,700	17,280	19,600	
2 1/4	3.547	31,880	38,250	46,750	14,100	17,000	18,800	20,730	23,600	
2 1/2	3.976	33,750	40,500	49,500	16,800	20,100	22,400	24,600	28,000	
2 3/4	4.430	35,630	42,750	52,250	19,700	23,700	26,300	28,900	32,900	
3	4.909	37,500	45,000	55,000	23,000	27,600	30,700	33,700	38,400	
3 1/4	5.412	39,380	47,250	57,750	26,600	32,000	35,500	39,000	44,400	
3 1/2	5.940	41,250	49,500	60,500	30,600	36,800	40,800	44,900	51,000	
3 3/4	6.492	43,130	51,750	63,250	35,000	42,000	46,700	51,300	58,300	
4	7.069	45,000	54,000	66,000	39,800	47,700	53,000	58,300	66,300	
4 1/4	7.670	46,880	56,250	68,750	44,900	53,900	59,900	65,900	74,900	
4 1/2	8.296	48,750	58,500	71,500	50,600	60,700	67,400	74,100	84,300	
4 3/4	8.946	50,630	60,750	74,250	56,600	67,900	75,500	83,000	94,400	

3½	9.621	52,500	63,000	77,000	63,100	75,800	84,200	92,600	105,200
3½	10.32	54,380	65,250	79,750	70,100	84,200	93,500	102,900	116,900
3½	11.05	56,250	67,500	82,500	77,700	93,200	103,500	113,900	129,400
3½	11.79	58,130	69,750	85,250	85,700	102,800	114,200	125,600	142,800
4	12.57	60,000	72,000	88,000	94,200	113,100	125,700	138,200	157,100
4½	13.36	61,880	74,250	90,750	103,400	124,000	137,800	151,600	172,300
4½	14.19	63,750	76,500	93,500	113,000	135,700	150,700	165,800	188,400
4½	15.03	65,630	78,750	96,250	123,300	148,000	164,400	180,800	205,500
4½	15.90	67,500	81,000	99,000	134,200	161,000	178,900	196,800	223,700
4½	16.80	69,380	83,250	101,750	145,700	174,800	194,300	213,700	242,800
4½	17.72	71,250	85,500	104,500	157,800	189,400	210,400	231,500	263,000
4½	18.67	73,130	87,750	107,250	170,600	204,700	227,500	250,200	284,400
5	19.64	75,000	90,000	110,000	184,100	220,900	245,400	270,000	306,800
5½	21.65	78,750	94,500	115,500	213,100	255,700	284,100	312,500	355,200
5½	23.76	82,500	99,000	121,000	245,000	294,000	326,700	359,300	408,300
5½	25.97	86,250	103,500	126,500	280,000	335,900	373,300	410,600	466,600
6	28.27	90,000	108,000	132,000	318,100	381,700	424,100	466,500	530,200
6½	30.68	93,750	112,500	137,500	359,500	431,400	479,400	527,300	599,200
6½	33.18	97,500	117,000	143,000	404,400	485,400	539,200	593,100	674,000
6½	35.79	101,250	121,500	148,500	452,900	543,500	603,900	664,200	754,800
7	38.48	105,000	126,000	154,000	505,100	606,100	673,500	740,800	841,900
7½	41.28	108,750	130,500	159,500	561,200	673,400	748,200	823,000	935,300
7½	44.18	112,500	135,000	165,000	621,300	745,500	828,400	911,200	1,035,400
7½	47.17	116,250	139,500	170,500	685,500	822,600	914,000	1,005,300	1,142,500
8	50.27	120,000	144,000	176,000	754,000	904,800	1,005,300	1,105,800	1,256,600

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NOTE.—All items in this index refer first to the section and then to the page of the section. Thus, "Anchor bolts, §73, p33," means that anchor bolts will be found on page 33 of section 73. The letters "BT" stand for "Bridge Tables." These tables follow Section 73.

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